STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION

DRAINAGE HANDBOOK
STORM DRAINS

OFFICE OF DESIGN, DRAINAGE SECTION   OCTOBER 2014

TALLAHASSEE, FLORIDA

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Chapter 1 - Introduction

1.1 Background

The 1987 Florida Department of Transportation Drainage Manual was published as a three volume set: Volume 1 - Policy; Volumes 2A and 2B - Procedures; Volume 3 - Theory. On October 1, 1992, Volume 1 - Policy was revised to Volume 1 - Standards. With that revision, Volumes 2A, 2B, and 3 were designated as general reference documents. The Volume 1 - Standards was revised in January 1997 and was renamed to simply the “Drainage Manual”. No revisions have been, nor will be made to volumes 2A, 2B, and 3 of the 1987 Drainage Manual.

This handbook is one of several the Central Office Drainage section developed to replace Volumes 2A, 2B, and 3 of the 1987 Drainage Manual. In this form, the current Drainage Manual will be maintained as a “standards” document, while the handbooks will cover general guidance on FDOT drainage design practice, analysis and computational methods, design aids, and other reference material.

1.2 Purpose

This handbook is intended to be a reference for designers of FDOT projects, and to address issues with design, construction and maintenance of the Department’s storm drains. Pertinent sections of the 1987 Drainage Manual have been incorporated into this handbook.

The guidance and values provided in this handbook are suggested or preferred approaches and values, not requirements nor standards. The values provided in the Drainage Manual are the minimum standards. In cases of discrepancy, the Drainage Manual standards shall apply. As the Drainage Manual states about the standards contained in it, situations exist where the guidance provided in this handbook will not apply. The inappropriate use of and adherence to the guidelines contained herein, does not exempt the engineer from the professional responsibility of developing an appropriate design.

1.3 Distribution

This handbook is available for downloading from the Drainage Internet site.

1.4 Revisions

Any comments or suggestions concerning this handbook may be made by e-mailing the State Hydraulics Engineer

1.5 Definitions of Terms and Acronyms

Annulus	The area between the outside of a pipe and the precast opening in which the pipe is placed.
CFS	Cubic Feet per Second
Conveyance	A measure of the carrying capacity of a channel or pipe section. For the standard Manning's equation: Conveyance = $\frac{Q}{\sqrt{S}} = \frac{1.49}{n} AR^{2/3}$
Critical Depth (D_c)	The depth associated with the minimum total energy for a particular flow rate in a particular cross section. The flow depth can drop through critical depth at the outlet of a pipe section if the water surface downstream is low enough.
Design Tailwater (DTW)	The elevation of the hydraulic gradient (or water surface) at the outlet of a storm drain system during the design storm event.
FHWA	Federal Highway Administration
Full Flow Friction Slope	The slope obtained from Manning's equation using an area equal to the full cross sectional area of the pipe and a flow rate equal to the design flow rate. $S = [Qn /(1.49AR^{2/3})]^2$ where Q = design flow rate A & R = based on full cross section area of pipe For pipes flowing full, the Full Flow Friction Slope is recorded as the Hydraulic Grade Line Slope in the tabulation form.
Full Flow Friction Loss	This is calculated as: Full Flow Friction Loss = Full Flow Friction Slope x Pipe Length. For pipes flowing full, the full flow friction loss is recorded as the hydraulic gradient fall in the tabulation form.
Gutter Drain	A pipe, used along steep slopes, to convey stormwater from shoulder gutter inlets on elevated roadways to drainage conveyance systems below at a much lower elevation.
HEC	Hydraulic Engineering Circular. Produced by the FHWA.

Hydraulic Grade Line (HGL)	In open channel flow, it is the water surface along the channel reach. In pressure flow, it is a theoretical line connecting hydraulic gradient points along the flow path.
Hydraulic Gradient (HG)	The elevation of the water surface in open channel flow. In pressure flow it is the elevation to which the water would rise in a tube or inlet connecting the flow pipe to atmospheric pressure.
Lower End HG	The elevation of the hydraulic gradient at the downstream end of a pipe section.
Minor Losses	All losses that are not due to friction. Generally these are energy losses due to changes or disturbances in the flow path. Minor losses include such things as entrance, exit, bend, and junction.
Physical Velocity	The velocity in a pipe that is flowing full, but not under pressure. This condition is sometimes called gravity full flow and the velocity is determined from Manning's equation. $V = (1.49/n)\, R^{2/3}\, S_{PHYSICAL}^{1/2}$ where R is based on full cross section area of the pipe
Spread	The horizontal distance of the stormwater flowing down a pavement & gutter section from the face of the gutter to the water’s edge.
Tailwater	The hydraulic gradient (water surface elevation) downstream of a pipe section.
t_c	Time of Concentration. Refer to Section 2.2 for discussion.
Upper End HG	The elevation of the hydraulic gradient at the upstream end of a pipe section.

1.6 FDOT Storm Drain Tabulation Form

The primary means of documenting the storm drain design is the Department’s storm drain tabulation form shown in Figure 1-1. The items to be recorded on this form have been identified by numbers in parentheses in Figure 1-1 and are discussed in the description following the form. This information is also available on the FDOT Drainage Web Site.

FLORIDA DEPARTMENT OF TRANSPORTATION
STORM DRAIN TABULATION FORM

Financial Project Identification: County: Network: Prepared: Date:
Description: Organization: State Road: Checked: Date:

LOCATION OF UPPER END			STRUCTURE NO.	TYPE OF STRUCTURE	LENGTH (ft.)	DRAINAGE AREA (Acres)		SUB-TOTAL (C*A)	TIME OF CONCENTRATION (min)	TIME OF FLOW IN SECTION (min)	INTENSITY (in/hr)	TOTAL (C*A)	BASE FLOW (cfs) *	TOTAL FLOW (cfs)	MINOR LOSSES (ft.) ***	INLET ELEVATION (ft.)	HGL CLEARANCE (ft.) *	HYDRAULIC GRADIENT			NUMBER OF BARRELS *	PIPE SIZE (in.)	SLOPE (%)	ACTUAL VELOCITY (fps)	FULL FLOW CAPACITY (cfs) *	NOTES AND REMARKS
						C= **												CROWN								ZONE:
						C= **												FLOWLINE				RISE	HYD. GRAD.			FREQUENCY (Yrs):
ALIGNMENT NAME			UPPER			C= **																	PHYSICAL			MANNING'S "n":
STATION	DISTANCE (ft.)	SIDE	LOWER			INCREMENT	TOTAL											UPPER END ELEVATION (ft.)	LOWER END ELEVATION (ft.)	FALL (ft.)		SPAN	MIN. PHYS.	PHYSICAL VELOCITY (fps)		TAILWATER EL (ft):

* Denotes optional information.
** A composite runoff coefficient may be shown in lieu of individual C-values, provided the composite C calculations are included in the drainage documentation.
*** Required if Minor Losses are included

Figure 1-1

Tabulation Form Description:

1. Runoff Coefficients (C): You will be limited to three runoff coefficients. For most projects this provides sufficient flexibility.

2. Alignment Name: The name of the alignment that the structure's station and offset references.

3. Station: The survey station number for the structure being used.

4. Distance: The offset distance from reference point of the structure to the reference station.

5. Side: The side, right (Rt.) or Left (Lt.), of the reference station.

6. Structure Number: The structure number at the upper end is shown above the structure number at the lower end. Each major row (3 minor rows) of the form identifies an inlet and the downstream pipe from that inlet.

7. Type of Structure: Usually shown with abbreviations such as Type P-3 or P-5 for inlets; Type C or E for ditch bottom inlets (DBI); Type P-8 or J-7 (MH) for manholes; and Type J-7 (Junct) for junction boxes.

8. Length (ft): The length, in feet, from the hydraulic center of the structure to the hydraulic center of the next downstream structure.

9. Increment: The incremental drainage areas corresponding to the runoff coefficients being used. It is normally only the area that drains overland to an inlet, but it can include areas that drain to structures through existing pipes. If so, note it in the Remarks Column (#42) or use the optional Base Flow Column.

   Manholes usually do not have incremental areas as they are handling areas already tabulated. If the incremental drainage area does not fit one of the three-runoff coefficients selected, mathematically adjust the size of the area to fit one of the selected runoff coefficients. Note the adjustment in the Remarks Column (#42).

   $Area_{ADJ} = (C_{ACT} / C_{SELECT}) \times Area_{ACT}$

10. Total: The total area associated with each runoff coefficient and passing through the structure. Identify all the areas that drain to the structure through pipes from upstream structures. Add these "upstream areas" to the incremental drainage areas for the structure (# 9).

11. Subtotal (CA): The result of multiplying the total area associated with each runoff coefficient (#10) by the corresponding runoff coefficient.

12. Time of Concentration (Min): Usually the time required for the runoff to travel from the most hydraulically remote point of the area drained to the point of the storm drain system under consideration. This time consists of overland flow, gutter flow, and flow time within the pipe system. Occasionally this time is associated with a reduced area that creates a peak flow. If so, note it in the Remarks Column (#42). Show in minutes.

13. Time of Flow in Section (min): The time, in minutes, it takes the runoff to pass through the section of pipe.
    T_{SECT}=hydraulic length (#8)/actual velocity (#35)

14. Intensity: Determined from one of the eleven Intensity-Duration-Frequency (IDF) curves developed

by the Department. Intensity depends on the design frequency and the time of concentration. Show in inches per hour.

15. Total (CA): The sum of the subtotal CA values. (#11)

16. Base Flow (cfs): This is an optional column to account for known flows from underdrains, offsite pipe connections, etc. Show in cubic feet per second.

17. Total Flow (cfs): The product of the intensity (#14) and the Total CA (#15) plus Base Flows (#16). Show in cubic feet per second.

18 Minor Losses (ft): This is an optional column to account for minor losses according to section 3.6.2 of the Drainage Manual. Show to one hundredth of a foot.

19. Inlet Elevation: The elevation of the edge of pavement for curb inlets (Index 210 through 216). The elevation of the theoretical grade point for barrier wall Inlets of Indexes 217 & 219. The grate elevation as shown in the Indexes for barrier wall inlet (Index 218) and gutter inlets (Indexes 220 & 221). The grate elevation for ditch bottom inlets (Indexes 230 through 235). The elevation of the manhole cover for manholes.

20. HGL Clearance (ft): This is determined by the difference between the Inlet Elevation (#19) and the Upper End Hydraulic Gradient Elevation (# 21). Show to one hundredth of a foot.

21. Upper End (Hydraulic Gradient): The elevation of the hydraulic gradient at the upper end of the pipe section. The elevation, under design conditions, to which water will rise in the various inlets and manholes. Show to one hundredth of a foot.

22. Lower End (Hydraulic Gradient): The elevation of the hydraulic gradient at the lower end of the pipe section. Show to one hundredth of a foot.

23. Upper End (Crown Elevation): The inside crown elevation at the upper end of the pipe section. Show to one hundredth of a foot.

24. Lower End (Crown Elevation): The inside crown elevation at the lower end of the pipe section. Show to one hundredth of a foot.

25. Upper End (Flow Line): The flow line at the upper end of the pipe section. Show to one hundredth of a foot.

26. Lower End (Flow Line): The flow line at the lower end of the pipe section. Show to one hundredth of a foot..

27. Fall: The elevation change of the hydraulic grade line from the upper end to lower end of the pipe section. Show to one hundredth of a foot.

28. Fall: The physical fall of the pipe section. Show to one hundredth of a foot.

29. Number of Barrels: This optional column should be used for systems with pipe segments that have multiple barrels.

30. Pipe Size (Rise) (in.): The vertical distance between the Flow Line (#25) and the Crown (#23) in inches.

31 Pipe Size (Span) (in.): The horizontal distance of the inside of a pipe at its widest point in inches.

32. Slope (Hydraulic Gradient): For pipes under pressure flow this is the full flow friction slope. For pipes flowing partially full, this is: [Upper End HG (#21) - Lower End HG (#22)] / Hydraulic Length (#8). Show to one hundredth of a percent

33. Slope (Physical): Determined from Physical Fall (#28) / Hydraulic length (#8). Show to one hundredth of a percent.

34 Slope (Minimum Physical): The flattest physical slope to maintain a velocity of 2.5 FPS flowing full, obtained from rearranging Manning's equation:
$S_{MIN} = [Vn / (1.49R^{2/3})]^2$
Show to one hundredth of a percent.

35. Actual Velocity: Determined by Total Flow (#17) divided by the average cross-sectional flow area. See discussion in Chapter 5. Show to a minimum of one tenth of a foot per second.

36. Design Velocity: the Actual Velocity the pipe experiences under desing conditions, not necessarily a theoretical Manning's velocity created by the physical slope of the pipe.

37. Physical Velocity: The velocity produced when the pipe is flowing full based on the Physical Slope (#33). Show to a minimum of one tenth of a foot per second.

$V = (1.49/n)R^{2/3}S_{PHYSICAL}^{1/2}$

38. Full Flow Capacity (cfs): This optional column is the product of the Physical Velocity (#36) and the cross-sectional area of the pipe. Show in cubic feet per second.

39. Zone: One of the eleven FDOT Rainfall Zones published in the FDOT Hydrology Handbook.

40. Frequency: The Storm Drain Design Frequency according to Section 3.3 of the Drainage Manual.

41: Manning's "n": For Storm Drains this value should be 0.012 according to section 3.6.4 of the Drainage Manual. Document any other Manning's "n" values used in the Remarks Column (#42).

42. Tailwater El. (ft): The water elevation coincident with the outlet pipe and established by section 3.4 of the Drainage Manual. Some districts may have more stringent criteria.

43. Remarks: Include such things as: Area adjustments, partial flow depths, existing pipe connections, or anything unusual.

Chapter 2 - Hydrology

The rational method is used for pipe sizing, inlet capacity, and spread calculations.

$$Q = C\ i\ A$$

where:
- Q = Runoff in cubic feet per second (cfs)
- C = Runoff Coefficient (see Table 2-2 at the end of chapter)
- i = Rainfall intensity in inches per hour
- A = Area in acres

2.1 Design Frequency

The Drainage Manual states the design frequency for storm drains. For the Department's facilities, the frequencies range from 3-year to 50-year, with the most common being 3-year. These frequencies apply to pipe hydraulics, not inlet capacity nor spread within the roadway. The criteria for inlet capacity and spread are discussed in the next chapter. If a storm drain system includes both curb inlets and ditch bottom inlets, the ditch bottom inlets should be checked for a 10-year design frequency and all structures in the mixed system should meet the 3-year design frequency.

2.1.1 Storms of Greater Magnitude

You should always consider the intent of the Department's criteria regarding the flooding of properties upstream or downstream of Department's right of way. In several chapters of the Drainage Manual it says that any increases over pre-development stages shall not significantly change land use values. So you should consider the impacts of storm events that are more severe than the standard design frequency of the storm drain. Initially this should be a qualitative evaluation. Realize that there are several reasons why urban typical sections with storm drains can handle storms of greater magnitude.

The first is conservatism within the storm drain design procedure. The flow rate calculated for each pipe section is the peak flow rate. This is conservative because we calculate the hydraulic gradient assuming that peak flow rates exist in all of the pipe sections simultaneously. In reality, when one pipe section is at peak flow, usually one or more of the other pipe sections have flow rates less than peak. This is most evident when considering the differences between the upper and lower parts of a long system. For example, consider a system where the outlet pipe's flow is calculated based on a Time of Concentration of 35 minutes. The flow rates of the first several pipes were based on Times of Concentration of 10-15 minutes. If a 35 minute storm and its associated intensity is applied to the entire system, the flow rates in the first several pipes would be less than the flow rate we calculate based on Times of Concentration = 10-15 minutes. Therefore the friction losses in these pipes are actually less than we calculate. Conversely, during short intense storms the upper pipes could reach their design flow rates, but the downstream portion of the system does not have the entire area contributing, so downstream pipes do not see the design flows. This conservatism exists to some degree throughout all pipe system but has a minimal effect on short systems where the differences in Times of Concentration are small.

Another reason an urban typical section can handle storms of greater magnitude is that the roadway itself can convey substantial flow. A standard pavement section of 0.02 cross slope on a 0.3% longitudinal grade can convey approximately 7 cfs[1] with the depth of the flow at the top of the curb.

The last reason, although less significant, is that when the flow in the road reaches the height of the curb there is more pressure on the piping system, thus forcing more flow through the pipes.
Considering these reasons, look at the system to see if there are any places where the water elevations or discharge rates could be increased.

- Are there sags in the profile? If so, could the pond water leave the right of way at these locations? Would water have gone that direction in the pre-developed condition?

- Is the roadway blocking overland flow in any areas? If so, would the blocked water substantially change land use values?

- Where back of sidewalk inlets are used, should check valves or flap gates be used to prevent the water in the pipes from backing off of the right of way?

- Would the inlets at the ends of the system bypass flow during a more severe storm event? If so, would water have gone that direction in the pre-developed condition?

If you have concerns after considering these, it may be appropriate to do a more detailed evaluation. Perhaps check the operation of the storm drain system with higher frequency (less frequent) storm. Perhaps the storage in the road and the pipes could be modeled. A more detailed model of the pre-developed conditions may be needed.

If after evaluating these situations, it is evident there would be increased discharge or increases over pre-developed stages that would significantly change land use values, document this. Then change the storm drain design as necessary to bring the stages down or to reduce the discharge. Use larger pipes where necessary. This is not saying use a higher design frequency for the storm drain system. Increasing pipe

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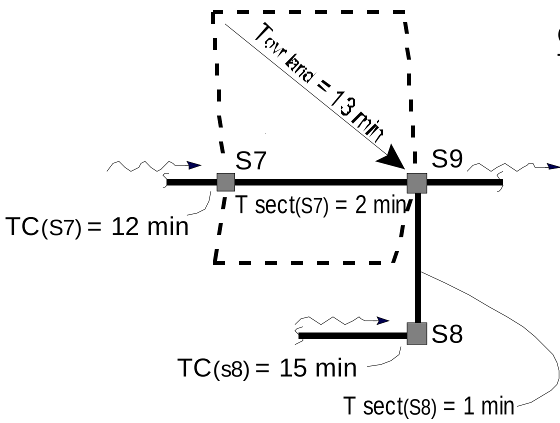
1 $Q = (0.56/n) \cdot S_x^{1.67} \cdot S^{0.5} \cdot T^{8/3} = (0.56/0.016) \cdot 0.02^{1.67} \cdot 0.003^{0.5} \cdot 18.75^{8/3} \approx 7$ cfs where T = curb height / cross slope = (4.5/12)/0.02 = 18.75’

sizes to prevent the adverse impacts to adjacent properties is different than using a higher design frequency and maintaining the standard hydraulic gradient clearance
.

2.2 Time of Concentration

The Time of Concentration (t_c) is the time required for the runoff to travel from the most remote point in the drainage basin to the point of the storm drain system under consideration. This will be the longer of: a) the overland flow time to the inlet or b) the sum of the t_c to the inlet immediately upstream in the piping system plus the time of flow through the upstream pipe section. For inlets that have more than one upstream pipe, you will need to compare the t_c and Time of Flow through Section of all the upstream inlets and pipes with the overland travel time to the subject inlet. Use the longest of these as the t_c. See Figure 2-1. For pipe segments that do not have upstream pipes the t_c will be simply the overland flow time.



Given the information shown:
TC(S9) is larger of:
a) T overland = 13 min

b) TC(S7) + Time of Flow in Section (S7)
12 min + 2 min = 14 min

c) TC(S8) + Time of Flow in Section (S8)
15 min + 1 min = 16 min

Therefore TC(S9) = 16 min

Determining Time of Concentration

Figure 2-1

2.2.1 Peak Flow from Reduced Area

Check to see if a portion of the drainage area will produce a larger flow rate than the entire area. This could occur where a larger portion of the drainage area exists towards the bottom or outlet as in Figure 2-2. This is even more likely if the land cover of the area towards the outlet is more impervious than the upstream area. Mathematically this is observed where the reduction in area is more than offset by an increased

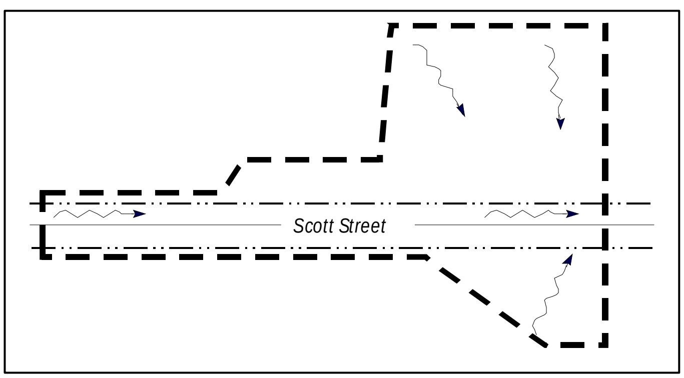


Figure 2-2

intensity and possibly an increased runoff coefficient.

The Department encourages that this check be made at apparent junctions or inlets in a storm drain system. It is acceptable but not necessary to check every pipe section for peak flow from reduced area. Some computer programs may do this automatically.

Example 2.1 - Peak Flow from a Reduced Area

Given:

- The partial Storm Drain system shown in Figure 2-3
- Project located in San Antonio, Pasco County, Zone 6

Find:

- The design flow rate for pipe section P_{31-32}.

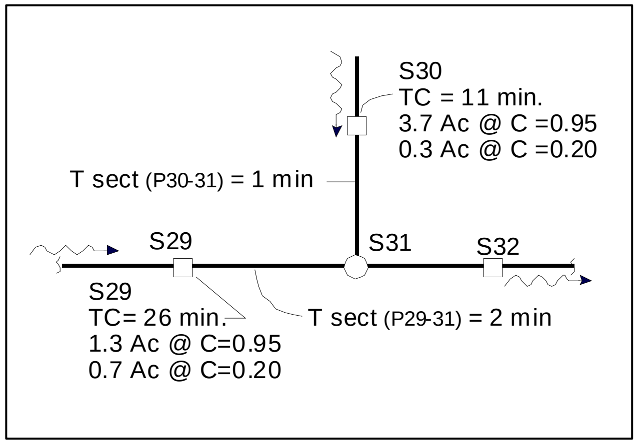


Figure 2-3 Example 2.1

First calculate the flow rate using the total drainage area (maximum t_c)

1. Add the product of CA for the upstream areas.
   Total CA_{S-29} = 0.95 x 1.3 ac + 0.2 x 0.70 ac = 1.38
   Total CA_{S-30} = 0.95 x 3.7 ac + 0.2 x 0.30 ac = 3.58
   Total CA_{S-31} = 4.96

2. Determine the time of concentration.

   The t_c is time for the entire drainage area to contribute. It is the longer of
   $(t_c)_{S-29}$ + Time of Flow in Section P_{29-31} = 26 + 2 = 28 min
   $(t_c)_{S-30}$ + Time of Flow in Section P_{30-31} = 11 + 1 = 12 min
   Therefore $(t_c)_{S-31}$ = 28 min.

3. Determine the intensity.
   From the IDF curve the intensity is 4.0 iph.

4. Determine the flow.

   Q = (CA) x i = 4.96 x 4.0 = 19.8 cfs

Now check for a larger flow from part of the drainage area. (Peak flow from reduced area.)

5. Determine the intensity associated with the shorter t_c.
   The shorter system time is from S-30 and is 11 + 1 min = 12 min.
   The intensity in Zone 6 for 12 min = 6.0 iph.

6. Estimate the area which will contribute from S-29 during a 12-minute storm.

   One approach is to reduce the area from the pipes having long times of concentration by the ratio of the times of concentrations. Ratio = (Short t_c) / (t_c of the associated pipe).

   $A_{S\text{-}29}$ is reduced by 12 min / 28 min = 0.43
   $A_{S\text{-}29\ REDUCED}$ @ C = 0.95 = 1.3 ac x 0.43 = 0.56 ac
   $A_{S\text{-}29\ REDUCED}$ @ C = 0.20 = 0.7 ac x 0.43 = 0.30 ac

7. Add the areas which will contribute to S-31 during a 12-minute storm.

   $Area_{TOTAL} = Area_{S\text{-}29\ REDUCED} + Area_{S\text{-}30}$
   @ 0.95 = 0.56 + 3.7 = 4.26
   @ 0.20 = 0.30 + 0.3 = 0.60

8. Add the product of CA contributing to S-31 during a 12-minute storm.
   Total CA = 0.95 x 4.26 + 0.2 x 0.6
   = 4.05 + 0.12 = 4.17

9. Determine the flow from the reduced area.

   $Q_{Reduced\ Area} = (C \times A) \times i_{12\ min}$
   $= 4.17 \times 6.0 = 25.0$ cfs

For pipe sections downstream of $P_{31\text{-}32}$, the incremental drainage areas would be added to the reduced areas recorded for $P_{31\text{-}32}$. The time of flow in downstream sections would be added to the reduced time of concentration for $P_{31\text{-}32}$.

A way of showing these approaches on the Tabulation form is shown in Table 2-1.

STRUCTU RE NUMBER	TYPE OF STRUCTURE	DRAINAGE AREA (ac.)		SUB TOTAL C•A	TIME OF CONCENTRATION (MINUTES)	TIME OF FLOW IN SECTION (MINUTES)	INTENSITY (IPH)	C•A TOTAL	TOTAL FLOW (cfs)	NOTES AND REMARKS	Example Approach
		C = 0.95								Zone:	
UPPER		C =								Frequency (Yrs):	
		C = 0.2								Manning's "n":	
LOWER		INCREMENT	TOTAL							Tailwater EL (ft):	
		0.2	1.3	1.24							Total Area – Max t_c
29	J1				26	2		1.38			
31		0	0.7	0.14							
		n/a	3.7	3.52							
30	J1				11	1		3.58			
31		n/a	0.3	0.06							
			5.0	4.75							
31	MH				28	-	4.0	4.95	19.8		
32			1.0	0.20							
		0.2	1.3	1.24							Peak Flow from Reduced Area
29	J1				26	2		1.38			
31		0	0.7	0.14							
		n/a	3.7	3.52							
30	J1				11	1		3.58			
31		n/a	0.3	0.06							
		0.2	4.26	4.05						Area from S-29 reduced by 12/28. Intensity based on System Time from S-30	
31	MH		0.0		12	-	6.0	4.17	25.0		
32			0.6	0.12							

Table 2-1

2.2.2 Ignoring Time of Flow in Section

For systems where the pipes are full without a storm event because of normal tailwater conditions, the time of flow in the pipe section is meaningless. In order for the runoff to get into the pipe, the water that is in the pipe has to move out. Since water under the pressures we are dealing with is essentially incompressible, what goes in the inlet must be coming out the outlet at the same time. In these situations, it is realistic to ignore the travel time through pipes that are submerged by normal tailwater. Note that normal tailwater (perhaps the control elevation of a wet pond) not the design tailwater should be used to determine if a pipe segment is submerged.

The Department realizes that current design software does not use the approach of ignoring time of flow in section. As such, some districts may not require that time of flow be ignored through submerged pipes.

RUNOFF COEFFICIENTS [a]

Slope	Land Use		Sandy Soils Min.	Sandy Soils Max.	Clay Soils Min.	Clay Soils Max.
Flat (0-2%)	Woodlands		0.10	0.15	0.15	0.20
	Pasture, grass, and farmland [b]		0.15	0.20	0.20	0.25
	Bare Earth		0.30	0.50	0.50	0.60
	Rooftops and pavement		0.95	0.95	0.95	0.95
	Pervious pavements[c]		0.75	0.95	0.90	0.95
	SFR:	1/2-acre lots and larger	0.30	0.35	0.35	0.45
		Smaller lots	0.35	0.45	0.40	0.50
		Duplexes	0.35	0.45	0.40	0.50
	MFR:	Apartments, townhouses, and condominiums	0.45	0.60	0.50	0.70
	Commercial and Industrial		0.50	0.95	0.50	0.95
Rolling (2-7%)	Woodlands		0.15	0.20	0.20	0.25
	Pasture, grass, and farmland [b]		0.20	0.25	0.25	0.30
	Bare Earth		0.40	0.60	0.60	0.70
	Rooftops and pavement		0.95	0.95	0.95	0.95
	Pervious pavements [c]		0.80	0.95	0.90	0.95
	SFR:	1/2-acre lots and larger	0.35	0.50	0.40	0.55
		Smaller lots	0.40	0.55	0.45	0.60
		Duplexes	0.40	0.55	0.45	0.60
	MFR:	Apartments, townhouses, and condominiums	0.50	0.70	0.60	0.80
	Commercial and Industrial		0.50	0.95	0.50	0.95
Steep (7%+)	Woodlands		0.20	0.25	0.25	0.30
	Pasture, grass, and farmland [b]		0.25	0.35	0.30	0.40
	Bare Earth		0.50	0.70	0.70	0.80
	Rooftops and pavement		0.95	0.95	0.95	0.95
	Pervious pavements [c]		0.85	0.95	0.90	0.95
	SFR:	1/2-acre lots and larger	0.40	0.55	0.50	0.65
		Smaller lots	0.45	0.60	0.55	0.70
		Duplexes	0.45	0.60	0.55	0.70
	MFR:	Apartments, townhouses, and condominiums	0.60	0.75	0.65	0.85
	Commercial and Industrial		0.60	0.95	0.65	0.95

a. Weighted coefficient based on percentage of impervious surfaces and green areas must be selected for each site.

b. Coefficients assume good ground cover and conservation treatment.

c. Depends on depth and degree of permeability of underlying strata.

Note: SFR = Single Family Residential, MFR = Multi-Family Residential

Table 2-2

Chapter 3 - Inlets and Pavement Hydraulics

3.1 Inlets

Factors controlling the selection of an inlet type include such things as hydraulics, utility conflicts, right of way limits, bicycle and pedestrian safety, etc. Figures 3-11 and 3-12 provide guidelines for selecting inlets. These figures contain information formerly included on Design Standards numbers 209 and 229.

3.1.1 Apparent Locations

- Low points in the gutter. Double-throated inlets, such as Type 2, 4, & 6, are symmetrical about the centerline and are intended to accept flow from both sides. These are normally used where the minor gutter flow exceeds 50 feet in length or 0.5 cubic feet per second.

- Upstream of pedestrian cross walks.

- Upstream of curb returns (See Index 303).

- Twenty to twenty-five feet outside the flat cross sections in super elevation transitions. Though the flow may be small, the cross slope is nearly flat so the spread potential is high.

- Outside of driveway turnouts. If the adjacent property is to be developed or redeveloped, try to obtain the site plans to identify future driveway locations.

3.1.2 Sags

Normally one inlet at the low point in combination with inlets on each of the approaching grades is sufficient to meet spread criteria.

Use flanking inlets for sags that have no outlet other than the storm drain system such as underpasses, barrier wall sections, or depressed sections where the roadway is much lower than the surrounding ground. Flanking inlets are the inlets placed on each side of and fairly close to the sag inlet. They provide backup capacity for the sag inlet should it become clogged. Thus flanking inlets should be positioned to operate when the pond water reaches the height of the allowable spread as shown in Figure 3-1. Vertical curve formulae are provided in Figure 3-9 (end of chapter) to help determine the flanking inlet locations.

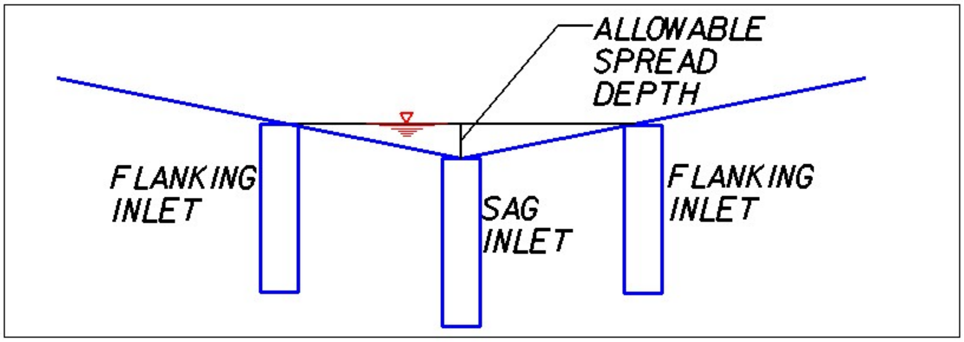


Figure 3-1

3.1.3 Continuous Grades

The initial placement of curb and gutter inlets on a continuous grade should be based on the 300' maximum spacing. After the initial placement of inlets, check the spread and add or move inlets as necessary to meet the spread standards.

The piping system layout may affect the locations of curb and gutter inlets. As you lay out the piping system you may find the need for a manhole to redirect the flow, or to provide maintenance access, or merely to connect stub pipes. If you use an inlet rather than a manhole, you get the benefit of an additional hydraulic opening for little or no additional costs. Piping system layout is discussed in the next chapter.

3.1.4 Back of Sidewalk

Back of sidewalk inlets should be located where concentrated flows drain toward the road and where the proposed sidewalk would block overland flow. Often these areas are identified from the survey, the back of sidewalk profiles, and the proposed cross sections. Do not rely on these alone**!** Walk the entire project looking for areas where concentrated runoff flows to the road and for localized depressed areas that were not identified in the survey. Development may have changed the existing ground line since the time the survey was done. Your field review with the back of sidewalk profiles, and proposed cross sections will identify areas where back of sidewalk inlets are needed.

The Field Review is Critical
to Designing
Back of Sidewalk Drainage

Where many back of sidewalk inlets are needed, check with the roadway designer about modifying the roadway profile grade to better accommodate overland flow.

Design Standards number 282 contains the standard back of sidewalk drainage inlets. Yard drains and the double 4" pipes under the sidewalk should be used to correct small existing flooding problems. For any other back of sidewalk drainage, obtain right of way as necessary to construct a ditch bottom inlet or other substantial back of sidewalk drainage conveyance.

Where back of sidewalk inlets are connected to the department's storm drain system, check the hydraulic grade line elevation at these inlets to see if water would back up or leave the system causing adverse impacts to adjacent properties. If so, first consider increasing the size of some downstream pipe sections. If avoiding adverse impacts by increasing pipe sizes is not feasible, consider using check valves or flap gates in the pipe connected to the back of sidewalk inlet (see Figure 3-2). Flap gates and check valves are not desirable because they require maintenance, nevertheless, they may be the most practical option for some situations.

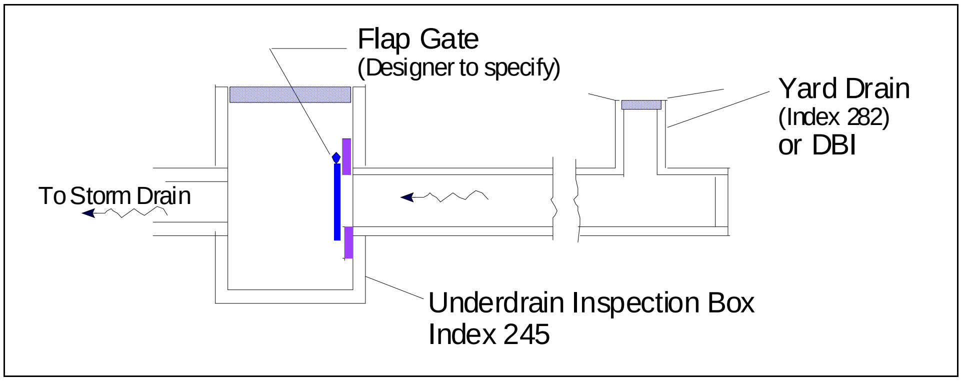


Figure 3-2

3.1.5 Inlet Capacity

Capacity data for most of the Department's inlets were developed by laboratory studies done at the University of South Florida (Anderson, 1972). A graphical presentation of this data is given in Appendix A. Separate curb inlet capacity charts are presented for various cross slopes. Methods described in USDOT, FHWA, Hydraulic Engineering Circular HEC-12 or 22 can also be used to evaluate the interception capacity of the Department's inlets.

3.2 Pavement Hydraulics

The Department uses driver visibility as a basis for the spread standards. There is a rainfall intensity that reduces the driver's sight distance to less than the minimum stopping sight distance. Removing the water from the road for intensities greater than this serves no purpose. If a driver's sight distance is less than the minimum stopping sight distance when the driver sees an object, the driver cannot stop in time regardless of how much water is on the road. So removing the water from the roads for intensities greater than the above intensity is over design from a vehicle standpoint.

The Department uses 4 inches per hour (iph) as the intensity that reduces the driver's sight distance to less than the minimum stopping sight distance. This is based on information summarized in FHWA HEC-21.

Use the integrated form of Manning's equation to calculate spread in gutters.

$$Q = \frac{0.56}{n} S_x^{5/3} S_L^{1/2} T^{8/3}$$

Where:
- Q = Gutter flow rate (CFS)
- n = Manning's roughness coefficient. (See Table 3-2, end of chapter)
- S_x = Pavement Cross Slope, (ft / ft)
- S_L = Longitudinal Slope, (ft / ft)
- T = Spread, (ft)

A nomograph for solving this equation is provided in Figure 3-8 (end of chapter). This equation is intended for triangular gutter sections. The standard Type F curb forms a composite section when combined with the pavement cross slope. In most cases, it is reasonable to ignore the gutter depression and treat the flow section as a simple gutter formed by the cross slope of the road and the curb. Ignoring the gutter depression is conservative[2], but allows for debris buildup in the gutter. If determining the additional capacity of the gutter depression is necessary, use Figure 3-10 (end of chapter) or the procedures provided in FHWA's HEC-12 or 22.

3.2.1 Gutter Grades

Standard gutter grades should not be less than 0.3 percent. Some District Drainage Engineers will approve 0.2 percent gutter grade in very flat terrain. Use of a saw tooth profile can maintain minimum grades in very flat terrain.

To provide adequate drainage in sag vertical curves, maintain a minimum gutter grade of 0.3 percent down to the inlet at the low point. Without this, the flat longitudinal grade near the low point would cause the spread to be greater than allowable. Maintaining the minimum gutter grade up to the inlet increases the cross slope at the low point, thus providing additional drainage. To maintain the minimum gutter grade, develop and show special gutter grades in the plans.

Example 3.1 - Special Gutter Grade

Given:

---

2 The gutter depression can add approximately 31% to the conveyance of the flow section in cases where the pavement cross slope is 0.02 and the travel lane is adjacent to the gutter (i.e. allowable spread = 7.5', design speed = 45 mph). For the common situation of a 4' bike lane adjacent to the gutter (i.e. allowable spread = 11.5', design speed = 45 mph) the gutter depression can add approximately 13% to the conveyance.

- The sag vertical curve described in the figure below.

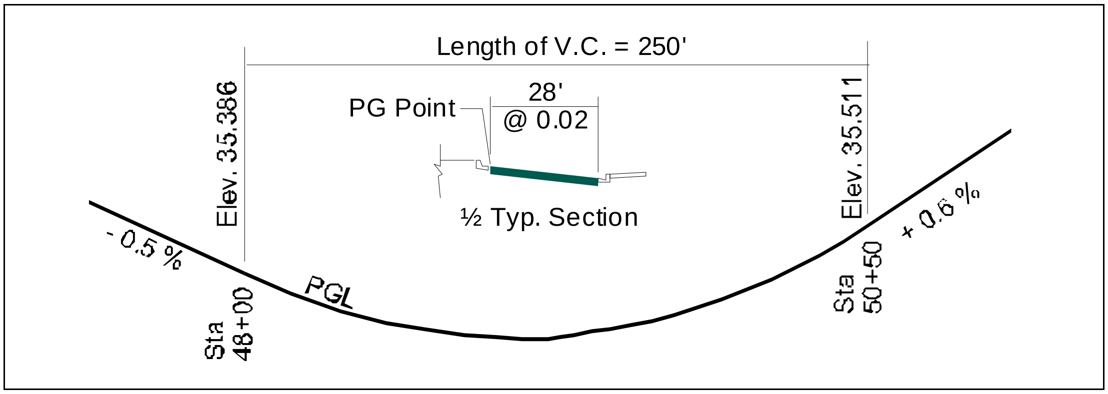


Figure 3-3 **Example 3.1 Given Information**

Find:

- The limits of the special gutter grade.
- The theoretical gutter elevation at the low point.
- The cross slope at the low point.

1. Determine the rate of change of longitudinal slope. (Formula from Fig. 3-9)

   rate of change = r = $(g_2 - g_1) / L$ = 0.6 -(-0.5) / 2.5 = 0.44

2. Find the location of the low point and the location where the longitudinal slope on the curve is -0.3% and +0.3%. Use the equation for longitudinal slope at any point and rearrange to solve for X.

$$\begin{aligned} X_{-0.3\,\%} &= (S_L - g_1) / r \\ &= [-0.3 - (-0.5)] / 0.44 = 0.4545 \text{ stations} \\ \therefore \quad \text{Station} &= \mathit{48{+}00} + \mathit{0{+}45.45} = 48{+}45.45 \end{aligned}$$

$$\begin{aligned} X_{+0.3\,\%} &= (S_L - g_1) / r \\ &= [0.3 - (-0.5)] / 0.44 = 1.8182 \text{ stations} \\ \therefore \quad \text{Station} &= \mathit{48{+}00} + 1{+}\mathit{81.82} = 49{+}81.82 \end{aligned}$$

Using the equation for the station of the turning point:

$$X_{\text{LOW POINT}} = (g_1 \times L) / (g_1 - g_2)$$
$$= (-0.5 \times 2.5) / (-0.5 - 0.6)$$
$$= 1.1364 \text{ stations}$$
$$\therefore \quad \text{Station} = \mathit{48{+}00} + 1{+}\mathit{13.64} = 49{+}13.64$$

So a special gutter grade of -0.3% is needed from Sta. 48+45.45 to Sta. 49+13.64 and a special gutter grade of +0.3% is needed from Sta. 49+13.64 to Sta. 49+81.82.

3. Find the elevation of the profile grade line at Sta. 48+45.45 and Sta. 49+81.82. Both are equal distance from the center so we only need to find one elevation.

$$Elev_{48+45.45} = Elev_{48+00} + g_1 X + ½ r X^2$$
$$= 35.386' + (-0.5)(0.4545) + ½ (0.44)(0.4545)^2$$
$$= 35.204'$$

4. Find the elevation at the gutter at Sta. 48+45.45 (This equals the elevation of the gutter at Sta. 49+81.82.)

The edge of pavement is 0.56' (28'x 0.02) lower than profile grade line and the gutter is 1.5" (0.125') below the edge of pavement so:

$Elev_{GUTTER} = Elev\ PGL_{Sta.\ 48+45.45} - 0.56 - 0.125 = 35.204' - 0.56' - 0.125' = 34.519'$

5. Find the theoretical gutter elevation at the low point.

$Elev = Elev_{Sta.\ 48+45.45}$ - (special gutter grade x length of special gutter)
$Elev = 34.519' - [0.3 \times (49.1364 - 48.4545)] = 34.314'$

This elevation would be used to check the hydraulic grade line clearance below the sag inlet.

6. Find the cross slope at the low point.

The elevation of the profile grade line at the low point is:

$$PGL\ Elev_{49+13.64} = Elev_{48+00} + g_1 X + ½ r X^2$$
$$= 35.386' + (-0.5)(1.1364) + ½ (0.44)(1.1364)^2$$
$$= 35.102'$$

The elevation at the edge of pavement at the low point is:

$$EOP\ Elev_{49+13.64} = Elev_{GUTTER} + 1.5''$$
$$= 34.314' + 0.125' = 34.439'$$

Cross Slope = (35.102' - 34.439') / 28' = 0.024 ft/ft

This would be used to check the spread of the inlet at the low. Interpolate between the values in Figures A-17 through A-19, where the cross slope value is between the values of the figures.

3.2.2 Cross Slope

Volume 1, Chapter 2 of the Plans Preparation Manual gives the standard cross slopes.

3.2.3 Shoulder Gutter

Shoulder gutter is used on fill slopes and at bridge ends to protect the slopes from erosion caused by water from the roadway and bridge. Shoulder gutter shall be used in accordance with section 3.7.3 of the Drainage Manual. Where placed at bridge ends, the gutter should be long enough to construct the gutter transitions shown on Design Standards numbers 400 and 220. The terminal shoulder gutter inlet should intercept all of the flow coming to it for a 10-year storm.

The Drainage Manual gives two spread criteria for sections with shoulder gutter. One is related to driver visibility (4 inches per hour) and the other is related to erosion protection of the fill slope (10-year). Both need to be met. Consider the potential for future additional lanes in the median when determining the flow rates in shoulder gutter.

In a typical situation where standard cross slopes and shoulder widths exist, the criterion for protecting the fill slope has a higher intensity and less allowable spread than the criterion for driver safety. Thus, the criterion for protecting the fill slope will set the inlet spacing.

Given the typical situation where both the shoulder and the miscellaneous asphalt behind the gutter slope upward at 0.06 from the gutter, the spread across the gutter and pavement section should not exceed 6' for the 10-year storm. This section has a conveyance of approximately 28 cubic feet per second [$K = Q / S_L^{1/2} = 28$ cfs]. The conveyance can be used to determine maximum allowable flow rates for various longitudinal slopes. Another approach is to treat the shoulder gutter and pavement section as a triangular gutter with a cross slope of 0.05, designing for 10 year flows, and limiting the spread to 6' (Figure 3-4).

1. The maximum shoulder gutter design conveyance should be K = 28 adjacent to guardrail, and K = 15 with no guardrail for the 10 year storm. K = 28 is derived from the flow area being limited to 15 inches outside the shoulder gutter and n = 0.016. K = 15 is derived from limiting the flow area to the shoulder gutter section.

2. The maximum shoulder gutter design conveyance approaching a terminal

gutter inlet should be K = 15 in order to intercept 100% of the design storm flow.

3. Consideration should be given to the placement of two gutter inlets at the down gradient shoulder gutter terminus in order to provide 100% interception, unless 100% interception by one inlet (K = 15) is demonstrated by appropriate calculation.

4. Inlets spacing shall meet spread criteria (DM, Sec. 3.9), maximum pipe length criteria (DM 3.10.1) and 10-year frequency gutter capacity criteria. In most cases, the 10-year frequency storm may govern inlet spacing.

5. Where applicable, inlet spacing shall be designed to accommodate the additional runoff from future widening.

6. Gutter inlet(s) should be placed at the down gradient end of all shoulder gutter, in lieu of concrete spillways or flumes, to reduce the potential for erosion.

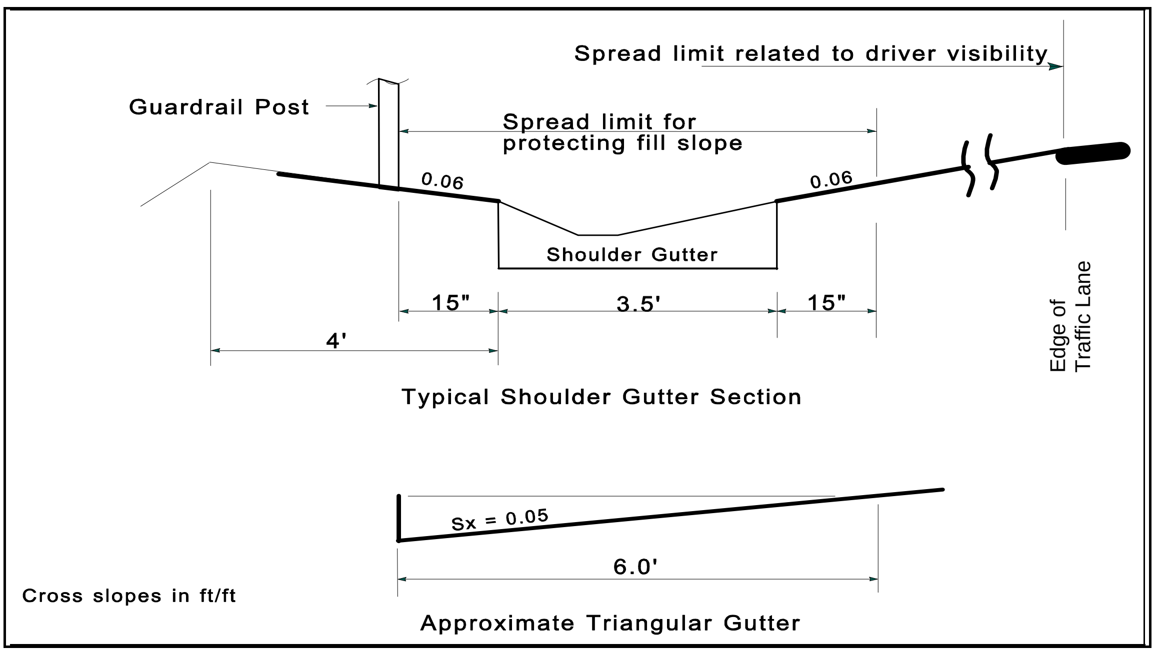


Figure 3-4

3.2.4 Determining the Spread

For roads that have uniform longitudinal grades and cross slopes, the spread calculations may be as simple as calculating the spread and bypass for the inlets with the largest overland flow. For these projects, you can usually make a reasonable assumption that if the inlets with the largest overland runoff do not exceed the spread standards and do not have any bypass, the other inlets will not exceed the spread standards and will not have any bypass. If you cannot comfortably make this assumption, the spread can be determined by the following procedure used with Table 3-1. In general, the information in Table 3-1 is the minimum required for spread calculations. Additional information can be provided and may be needed in certain situations.

Start at the upper most inlet and work to the low point, then start at the opposite high side and work back to the low.

1. Determine the drainage area and runoff coefficient of the overland runoff. Record the product of area and runoff coefficient (CA) in column 2.

2. Calculate the overland runoff by multiplying the product of CA in column 2 by the appropriate intensity (4 inches per hour or 10-year). $Q = C \bullet A \bullet i$.

3. Calculate the total flow to the inlet by adding the overland runoff in column 3 to the bypass from the upstream inlets.

4. Record the cross slope and longitudinal slope in columns 6 and 7 respectively.

5. Calculate the spread. If it is within standards, record in column 8 and go to the next step. If not, move the inlet (and add and move inlets as necessary) to make the spread acceptable and repeat steps 1 through 5.

6. Calculate intercepted flow and bypass flow. Record in column 9 and 10 respectively.

7. Proceed to the next downstream inlet and repeat steps 1 through 6.

FLORIDA DEPARTMENT OF TRANSPORTATION
SPREAD CALCULATIONS

Road:________________________________ Sheet ______of _______
County:______________________________ Prepared by:_____Date _____
Financial Project ID: ______________________ Checked by:_____Date _____
System Description: ____

Allowable Spread = _______
Manning's n = _______

Inlet No. or Location (1)	C • A (2)	Overland Runoff (3)	Previous By-pass (4)	Total Flow (5)	Cross Slope (ft/ft) (6)	Long Slope (%) (7)	Spread (8)	Intercepted Flow (9)	Bypass Flow (10)	Bypass to Inlet No. or to Inlet @ (11)

Table 3-1

Example 3.2 - Sag Vertical Curve

Given:

- The sag vertical curve and associated approach grades shown below.
- 4-lane Curb & Gutter section with 12' lanes, 12' continuous two-way left turn lane, 4' bike lane, and 6' sidewalk.
- Offsite drains to the road from 65' beyond the sidewalk.
- Offsite area draining to the road is impervious (C=0.95)
- Type 1 and 2 inlets are preferred by the District.
- Inlet location not restricted by driveways or side streets.
- Design Speed = 45 mph, then allowable spread is 11.5’ [1.5' gutter + 4' bike lane + 6' (½ of a travel lane)]
- A minimum gutter grade of 0.3% is used approaching the sag.

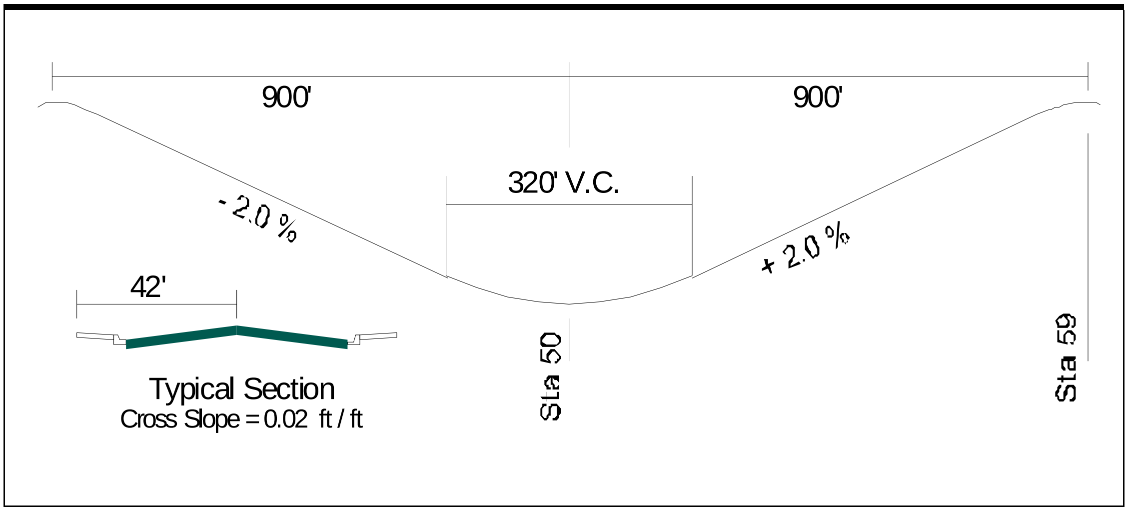


Figure 3-5
Example 3.2 Given Information

Find:

- Inlet spacing necessary to meet the spread criterion.

1. For the first try, place the inlets at the maximum 300' spacing out from the low. So the inlets will be placed at Station 44+00, 47+00, 50+00, 53+00, & 56+00.

The area to each inlet on the approach grades is:

Area = (½Rdwy Width + 65') x 300'
Area = (42'+65') x 300' / 43560 = 0.74 ac @ C=0.95
C x A = 0.95 x 0.74 = 0.70
$Q_{OVERLAND}$ = CAi
= 0.95 x 0.74 x 4 = 2.8 cfs.

2. Determine the spread, the intercepted flow, and the bypass (if any) for the uppermost inlets. (Sta. 44 & 56)

Spread (T) $= [(Q \times n) / (0.56 \times S_X^{5/3} \times S_L^{½})]^{3/8}$
This conservatively ignores the 1.5" gutter depression.

$= [(2.8 \times 0.016) / (0.56 \times 0.02^{5/3} \times 0.02^{½})]^{3/8}$

= 9.3' Acceptable (allowable spread is 11.5')
$Q_{INTRCEPT}$ ~ 2.1 cfs From Figure A-1.
Q_{BYPASS} = 2.8 - 2.1 = 0.7 cfs

3. Determine total flow to the next downstream inlets. (Sta. 47 & 53)

Q_{TOTAL} = $Q_{OVERLAND}$ +Q_{BYPASS}
= 2.8 + 0.7 = 3.5 CFS

4. Determine the spread, the intercepted flow, and the bypass.

Spread = 10.2' Still acceptable.
$Q_{INTRCEPT}$ = 2.3 cfs From Figure A-1.
Q_{BYPASS} = 3.5 - 2.3 = 1.2 cfs

5. Determine the spread approaching the sag inlet from either side.

Q_{TOTAL} = $Q_{OVERLAND}$ +Q_{BYPASS}
= 2.8 + 1.2 = 4.0 cfs

6. Determine the spread approaching the sag inlet. The longitudinal slope is 0.3% approaching the sag. For this example, the cross slope at the low is 0.021 ft/ft due to maintaining 0.3% gutter grade to the sag inlet. This was calculated using the approach in Example 3.1.

T $= [(4.0 \times 0.016) / (0.56 \times 0.021^{5/3} \times 0.003^{½})]^{3/8}$
= 14.7' Not acceptable.

The following table summarizes the above calculations.

Flow in cfs　　　Allowable spread = 11.5'　　　Manning's n = 0.016

1 Inlet Location (Sta.)	2 C • A	3 Overland Runoff	4 Previous By-pass	5 Total Flow	6 Cross Slope (ft/ft)	7 Long Slope (%)	8 Spread	9 Intercepted Flow	10 Bypass Flow	11 Bypass to Inlet @ Station
44+00	0.70	2.8	--	2.8	0.02	2.0	9.3	2.1	0.7	47+00
47+00	0.70	2.8	0.7	3.5	0.02	2.0	10.2	2.3	1.2	50+00
56+00	0.70	2.8	--	2.8	0.02	2.0	9.3	2.1	0.7	53+00
53+00	0.70	2.8	0.7	3.5	0.02	2.0	10.2	2.3	1.2	50+00
50+00 Approach	0.70	2.8	1.2	4.0	0.021	0.3	14.7	← Exceeds Standard		

7. Add and adjust inlets.

There is no direct solution. It is a trial and error process of moving inlets to reduce the spread. Adding an inlet to each side of the sag and adjusting the spacing of the inlets on the continuous grades should reduce the flow to the sag inlet and reduce the spread. Let us try placing the inlets at Stations 44+30, 46+80, 48+80, 50+00, 51+20, 53+20, & 55+70.

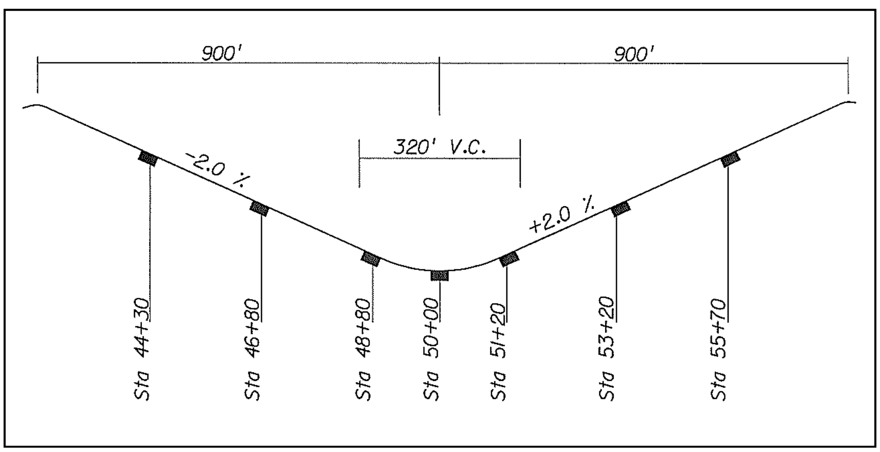


Figure 3-6

Example 3.2 - Second Iteration

The drainage area to the first continuous grade inlets (43+30, 55+70) is:

Area = (½Rdwy Width + 65') x 250'
Area = (42'+65') x 330' / 43560 = 0.81 ac @ C=0.95

The drainage area to the next inlets (46+80, 53+20) is:

Area = (42'+65') x 250' / 43560 = 0.61 ac @ C=0.95

The drainage area to the next inlets (48+80, 51+20) is:

Area = (42'+65') x 200' / 43560 = 0.49 ac @ C=0.95

The drainage area to each side of the sag is:

Area = (42'+65') x 120' / 43560 = 0.29 ac @ C=0.95

The inlets at stations 48+80 and 51+20 are on the vertical curve; therefore the longitudinal slope is flatter than 2.0%. Using vertical curve formulae:

Rate of change of grade (r) $= (g_2 - g_1) / L$
$= [2 - (-2)] / 3.2 = 1.25$ ft / station

Long slope $= g_1 + r X$ (X is dist along curve in Sta.)
$= -2 + 1.25 (0.4)$
$= -1.5\%$

For this example, the cross slope at the low is 0.021 ft/ft due to maintaining 0.3% gutter grade down to the sag inlet. This was calculated using the approach in Example 3.1. Using Figure A-17 (cross slope = 0.02) will provide a slight conservatism.

The following table shows the results of the change.

Flow in cfs Allowable Spread = 11.5 ft Manning's n = 0.016

1 Inlet Location (Sta)	2 CxA	3 Overland Runoff	4 Previous By-Pass	5 Total Flow	6 Cross Slope (ft/ft)	7 Long Slope (%)	8 Spread (ft)	9 Intercepted Flow	10 Bypass Flow	11 Bypass to Inlet @
44+30	0.77	3.1	0	3.1	0.02	2	9.7	2.2	0.9	46+80
46+80	0.58	2.3	0.9	3.2	0.02	2	9.8	2.2	1.0	48+80
48+80	0.47	1.9	1.0	2.9	0.02	1.5	9.9	2.4	0.5	50+00
50+00	0.28	1.1	0.5	1.6	0.021	0.3	10.5	n/a	n/a	n/a
55+70	0.77	3.1	0.0	3.1	0.02	2	9.7	2.2	0.9	53+20
53+20	0.58	2.3	0.9	3.2	0.02	2	9.8	2.2	1.0	51+20
51+20	0.47	1.9	1.0	2.9	0.02	1.5	9.9	2.4	0.5	50+00
50+00	0.28	1.1	0.5	1.6	0.021	0.3	10.5	n/a	n/a	n/a
50+00	0.56	2.2	1.0	3.2	0.021	n/a	6.3	n/a	n/a	n/a

In an actual project, the inlet location is affected by driveways and side streets.

Example 3.3 - Shoulder Gutter

Given

- The bridge approach grades shown below.
- 4-lane rural divided highway, 2-12' lanes, 10' paved outside shoulder, 4' sloped to gutter under guardrail (3' paved)
- Cross slope of shoulder and asphalt under guardrail = 0.06 ft/ft
- Fill slope is 10' high at station 67+00
- Project located in Zone 7, 10 year-10 min intensity = 7.4 in/hr
- Additional lanes may be added in future
- Runoff from bridge = 0.2 cfs (scuppers used on bridge)

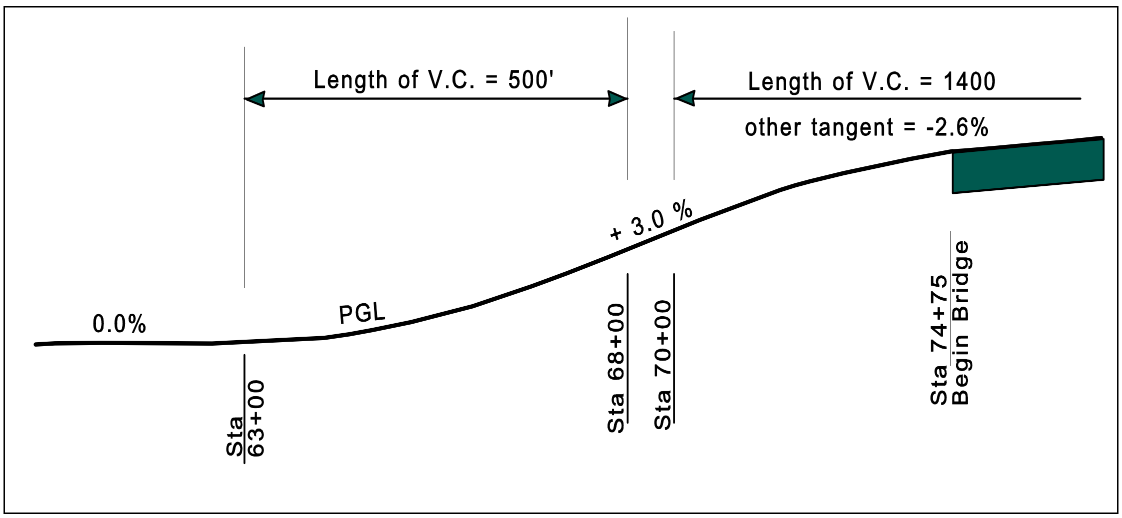


Figure 3-7

Example 3.3 Given Information

Find:

1. The location of the shoulder gutter inlets.

2. Determine the vertical curve geometry.

   Crest Curve:
   Rate of change of curve (r) = $(g_2 – g_1)/L$ = (-2.6 – 3.0)/14 = -0.4
   Long Slope at any point X = $g_1 + r X$ = 3.0 - 0.4 X

   Sag Curve:
   Rate of change of curve (r) = $(g_2 - g_1) / L$ = (3.0 - 0.0) / 5 = 0.6
   Long Slope at any point X = $g_1 + r X$ = 0.0 + 0.6 X

3. Estimate the lowest point at which shoulder gutter is needed.

   Shoulder gutter should be used on all fill slopes greater than 10' if the roadway grade is greater than 2%. For this example, the fill is approximately 10' at station 67+00. The longitudinal slope at this station 67+00 is: 0.6 x (67 - 63) = 2.4%. Since this is steeper than 2%, shoulder gutter should begin at or before station 67+00.

4. For the first try at inlet spacing, divide the distance between station 67+00 and the beginning of the bridge into equal distances that are less than 300'.
   Distance = 74+75 - 67+00 = 775'

   This equates to 3 spaces at approximately 258'. We will round it to 260', so the first inlet will be located at 74+75 - 260' = 72+15. The other inlets are at 69+55 and 66+95.

5. Determine the longitudinal slope at these inlets:

   @ 72+15 the longitudinal slope = 3 - 0.4 (72+15 - 70+00) = 3 - 0.4 x 2.15 = 2.14%
   @ 69+55 the longitudinal slope = 3%
   @ 66+95 the longitudinal slope = 0.6 x (66+95 - 63+00) = 0.6 x 3.95 = 2.37%

6. Determine area and overland runoff to each inlet.
   An additional lane may be added toward the median in the future so use 36' of pavement.
   Width = travel lanes + shoulder + gutter + slope* under guardrail.
   *Conservatively assume that all 4' sloping back to gutter is paved.

   = 36 +10 + 3.5 + 4 = 53.5'

   Area = 260' x 53.5' = 0.32 ac.
   C x A = 0.95 x 0.32 =0.30
   The travel time for flow along 260' of pavement is small so we will use the 10-minute intensity for the 10-year storm. i = 7.4 iph
   Q = CiA = 0.95 x 7.4 x 0.32 = 2.2 cfs

7. We will approximate the shoulder gutter as a triangular section with a cross slope of 0.05 ft/ft & n = 0.016. The spread must be limited to 6.0' in this triangular section to match the capacity of the shoulder gutter section. See previous discussion of shoulder gutter.

   Spread (T) $= [(Q \times n) / (0.56 \times S_X^{5/3} \times S_L^{1/2})]^{3/8}$

   The intercepted flow is determined from Figure A-16. The following table summarizes the calculations.

All flows (cfs) based on 10-year flow Allowable Spread = 6 ft Manning's n = 0.016

1 Inlet Location (Sta.)	2 C • A	3 Overland Runoff	4 Previous By-pass	5 Total Flow	6 Cross Slope (ft/ft)	7 Long Slope (%)	8 Spread	9 Intercepted Flow	10 Bypass Flow	11 Bypass to Inlet @ Station
72+15	0.30	2.2	0.2	2.4	0.05	2.14	4.9	2.4	---	
69+55	0.30	2.2	---	2.2	0.05	3.0	4.5	2.2	---	
66+95	0.30	2.2	---	2.2	0.05	2.37	4.7	2.2	---	

This inlet spacing meets the spread criterion for protecting the fill slopes and the last inlet captures all the runoff coming to it. Therefore, this design is acceptable. There is no need to check the 4 inches per hour criterion because the intensity would be less and the allowable spread would be greater.

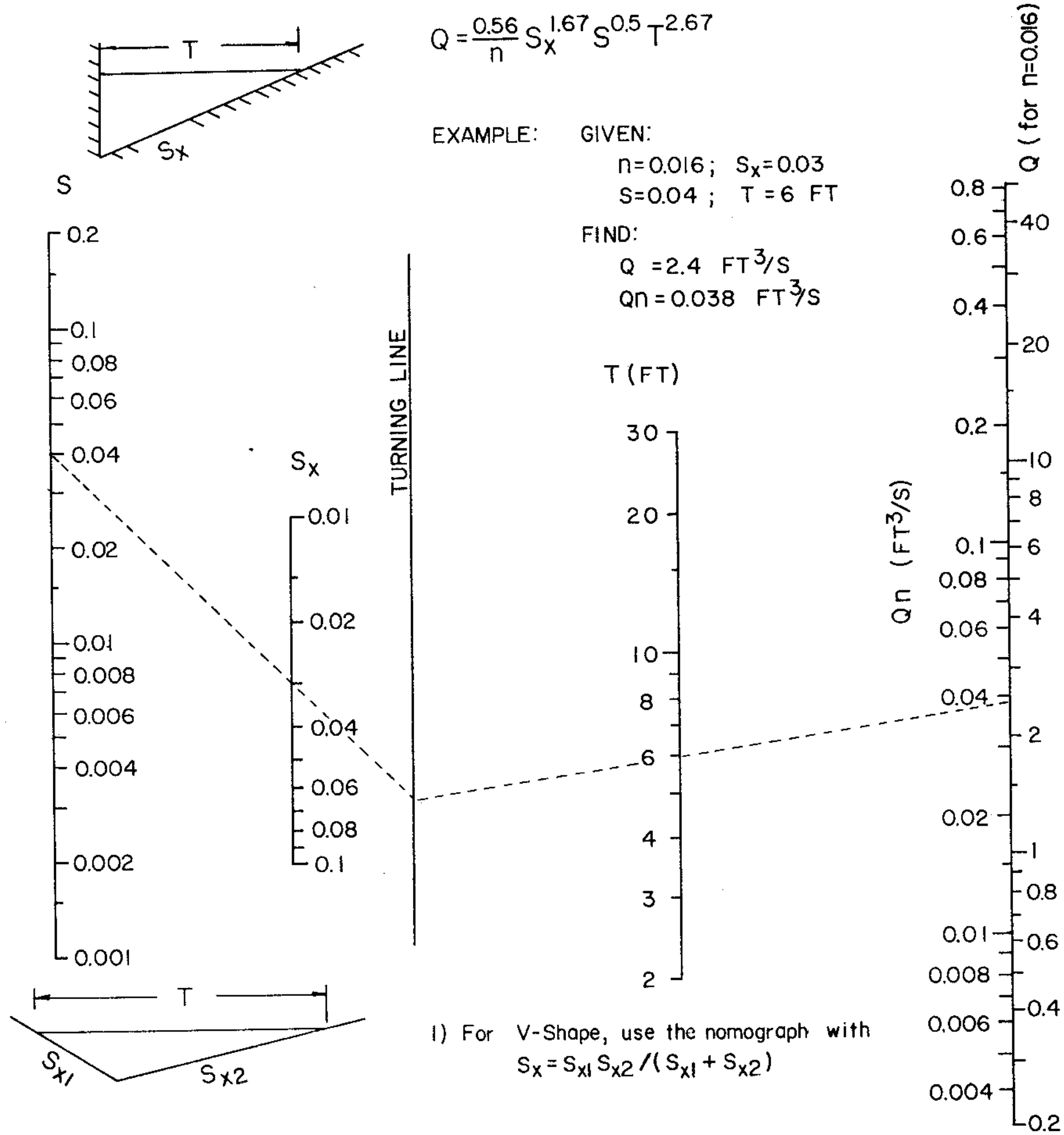
Manning's "n" Values for Street and Pavement Gutters

Type of Gutter or Pavement	Manning's n
Concrete gutter, troweled finish	0.012
Asphalt pavement:	
Smooth texture	0.013
Rough texture [1]	0.016
Concrete gutter with asphalt pavement:	
Smooth texture asphalt	0.013
Rough texture asphalt	0.015
Concrete pavement:	
Float Finish	0.014
Broom Finish [2]	0.016
For gutters with small slope, where sediment may accumulate, increase above values of "n" by	0.002

Reference: FHWA HEC-22

[1] The Department's friction course is rough texture asphalt.
[2] The Department's standard is brush (broom) finish for concrete curb. [Specification Section 520]

Table 3-2



Reference: FHWA Hec-12 (1984)

Figure 3-8

VERTICAL CURVE FORMULAE

1. Rate of Change of Grade: $r = \frac{g_2 - g_1}{L}$

2. Offset from tangent to curve: $y = \frac{1}{2} r X^2$

3. Elevation for any point on curve: $E_x = E_{pvc} + g_1 X + (\frac{1}{2} r X^2)$

4. Grade (longitudinal slope) at any point: $\frac{\partial E_x}{\partial X} = g_1 + rX$

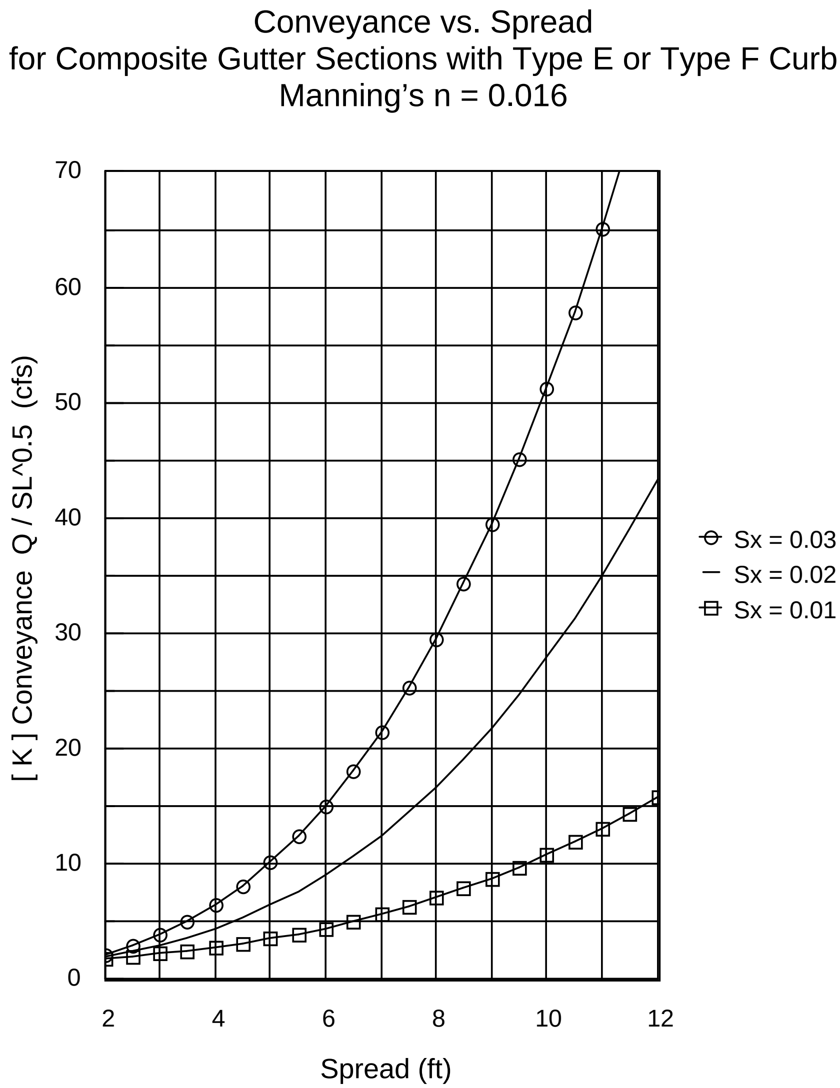
5. Station from PVC to turning point (local tangent horizontal) on a curve:

$$X = \frac{g_1 L}{g_{1-} g_2}$$

6. Elevation of turning point: $E_{TP} = E_{pvc} - \frac{1}{2}\left[\frac{L g_1^2}{g_2 - g_1}\right]$

Where:
All horizontal dimensions (X) are in Stations.
All vertical dimensions (E) are in Feet.
All grades are in percent.
L = Length of vertical curve in Stations.
E_{pvc} = Elevation of the Point of Vertical Curve.

Figure 3-9



Based on FHWA HEC-12, App. C.

S_L = Longitudinal Slope (ft/ft)
S_x = Pavement Cross Slope (ft/ft)

Example
Given: Q = 3 cfs, S_x = 0.03, S_L = 0.04 ft/ft

Find Spread:

1. $K = Q/S_L{}^{0.5} = 3 / 0.04^{0.5} = 15$
2. From Chart at K = 15 & S_x = 0.03, Spread = 6.0 ft

Figure 3-10

CURB INLET and GUTTER INLET APPLICATION GUIDELINES

INDEX NO	INLET TYPE	TYPE CURB/GUTTER	GRADE CONSIDERATION	BICYCLE COMPATIBLE	ACCEPTABLE IN PEDESTRIAN WAY	ACCEPTABLE IN AREAS OF OCCASIONAL PEDESTRIAN TRAFFIC	Notes	UTILITY LOCATION FROM CURB
210	1	E & F	Continuous	Yes	No	Yes		Inside
	2 [1]	E & F	Sag	Yes	No	Yes		Inside
	3	E & F	Continuous	Yes	No	Yes		Inside
	4 [1]	E & F	Sag	Yes	No	Yes		Inside
211	5	E & F	Continuous	Yes	No	Yes		Outside
	6 [1]	E & F	Sag	Yes	No	Yes		Outside
212	7	Separator I & II	Continuous or Sag	Yes	No	Yes		Inside
213	8	Separator IV & V	Continuous or Sag	Yes	No	Yes		Inside
214	9 [2]	D & F	Continuous or Sag	Yes	No	Yes		Outside
215	10 [2]	D & F	Continuous or Sag	Yes	No	Yes		Outside
217	1	Median Barrier Wall	Continuous	No	No	Yes [4]		N/A
	2 [1]	Median Barrier Wall	Sag	No	No	Yes [4]		N/A
	3	Median Barrier Wall [3]	Continuous	No	No	Yes [4]		N/A
	4 [1]	Median Barrier Wall [3]	Sag	No	No	Yes [4]		N/A
	5 [1]	Median Barrier Wall [3]	Continuous & Sag	No	No	Yes [4]		N/A
218	-	Barrier Wall	Continuous or Sag	No [5]	No	Yes	See Index 218 Inset B	N/A
219	-	Barrier Wall (Rigid, C & G)	Continuous or Sag	No [5]	No	Yes	See Index 219 Inset B & C	N/A
220	S	Shoulder	Continuous	No [5]	No	Yes	See Index 220 Bar Stub Detail	N/A
221	V	Valley	Continuous or Sag	No [5]	No	Yes		N/A

[1] Double throated inlets are usually not warranted unless the minor gutter flow exceeds 50 feet in length or 0.5 CFS.
[2] Curb Inlets 9 and 10 are to be used only where flows are light and right of way does not permit the use of throated curb inlets.
[3] These are double inlets; one on each side of the barrier wall.
[4] May be used by specifying the reticuline grate.
[5] Bicycle compatible provided a minimum 4ft riding surface is provided around the inlet, with a preferred 1 ft offset from the inlet. Consider use of pavement markings shown in the 2009 MUTCD to alert cyclist to the inlet in the bicycle lane or shoulder pavement.

Figure 3-11

DITCH BOTTOM AND MEDIAN INLET APPLICATION GUIDELINES

Index No.	Inlet Type	Location	Traffic	Bicycle Compatible	Acceptable in Pedestrian Way	Acceptable in Areas of Occasional Pedestrian Traffic [5]
230	A	Limited Access Facilities	Heavy Wheel Loads	No	No	No
231	B	Inside Clear Zone	Heavy Wheel Loads	No	No	Yes
232	C [3]	Outside Clear Zone [4]	Infrequent Traffic	Yes [6]	No	Yes [4]
	D	Outside Clear Zone [4]	Infrequent Traffic	Yes [6]	No	Yes [4]
	E	Outside Clear Zone [4]	Infrequent Traffic	Yes [6]	No	Yes [4]
	H	Outside Clear Zone	Infrequent Traffic	Yes	No	Yes
233	F	Inside Clear Zone	Heavy Wheel Loads	Yes	No	Yes
	G	Inside Clear Zone	Heavy Wheel Loads	Yes	No	Yes
234	J	Inside Clear Zone	Heavy Wheel Loads	No	No	Yes
235	K	Outside Clear Zone	N/A	N/A	No	N/A

[1] Alternate G grates should be specified when in salt-water environment.
[2] Inlets with slots are more debris tolerant than inlets without slots. Debris may buildup on Type B fence of Type K Inlet.
[3] For Back of Sidewalk Location See Index No. 282.
[4] Type C, D, & E Inlets without slots or inlets with traversable slots may be located within the Clear Zone. Slotted inlets located within the Clear Zone or in areas accessible to pedestrians shall have traversable slots.
[5] Areas subject to occasional pedestrian traffic are pavement, grassed, or landscaped areas where pedestrians are not directed over the inlet and can walk around the inlet.
[6] Inlets with traversable slots shall not be used in areas subject to bicycle traffic.

Figure 3-12

Chapter 4 - Pipe System Placement

4.1 Plan Layout

Once the inlets have been placed to drain the pavement adequately, lay out the piping system to connect the inlets. While laying out the system, you will add manholes as necessary to redirect the flow, or to provide maintenance access, or merely to connect stub pipes. At this stage consider adding an inlet instead of a manhole. When an inlet is used instead of a manhole, you get the benefit of an additional hydraulic opening for little or no additional cost.

There are several items to consider that can influence the piping system plan layout. The most important issues are hydraulics, constructability, and utility conflicts.

- Avoid placing pipes that would oppose flows from other pipes especially in high velocity situations. Impinging flows can be avoided by staggering the elevations of the pipes entering a junction box.

- Consider R/W necessary to open the trench for the pipes. This is especially important for deep pipes. Temporary sheet piling may be used during installation to reduce the trench width, but this is very costly and other alternatives (e.g. moving the trunk line) should be explored.

- Use either a manhole or an inlet at changes in flow direction. This is to provide maintenance access where debris and sediment often collect.

- Preferably, place manholes in or behind the sidewalk. This allows access without closing the travel lanes and is much safer for maintenance personnel. If manholes must be placed in the pavement, avoid putting the lids in the wheel path.

- Minimize interference with major utilities, such as fiber optic lines, and sanitary and potable water lines greater than 8 inches in diameter. See discussion in this chapter.

- Where there is one main trunk line, place it on the side of the road constructed first. This prevents constructing stub lines that can’t be drained.

- Where there is one main trunk line, locate it, if possible, on the low side of super elevated roadway sections to minimize the depth of cut.

- Where there is one main trunk line, consider connecting several inlets along the opposite side of the road from the trunk line, and then running only one pipe laterally across the road. This will reduce the number of cuts across the road.

- Consider using two trunk lines to minimize the number of cuts across the road and thus simplify the maintenance of traffic. In such cases, the gains in improved maintenance of traffic should be weighed against the increased cost of the additional trunk line.

4.1.1 Retaining Wall Drainage

Whenever possible, avoid placing piping within mechanically stabilized earth (MSE) retaining wall embankments, allowing stormwater to flow off of the MSE portion of the road and into inlets beyond the wall. This approach will avoid physical conflicts with the MSE strapping and the potential for leaking stormwater into the strap zone. If pipes must be placed within the MSE wall section, obtain concurrence from the District Drainage Engineer.

With the concurrence of the District Drainage Engineer, place trunk lines and the median structures at least two feet outside of the reinforced soil zone, as shown in Figure 4-3. Please refer to the following figures for preferred storm sewer layout, within MSE wall embankments. In the cases typified in Figures 4-1 and 4-2, diligently confer with the MSE wall structural designer to ensure that the structural integrity of the wall is preserved and is maintainable.

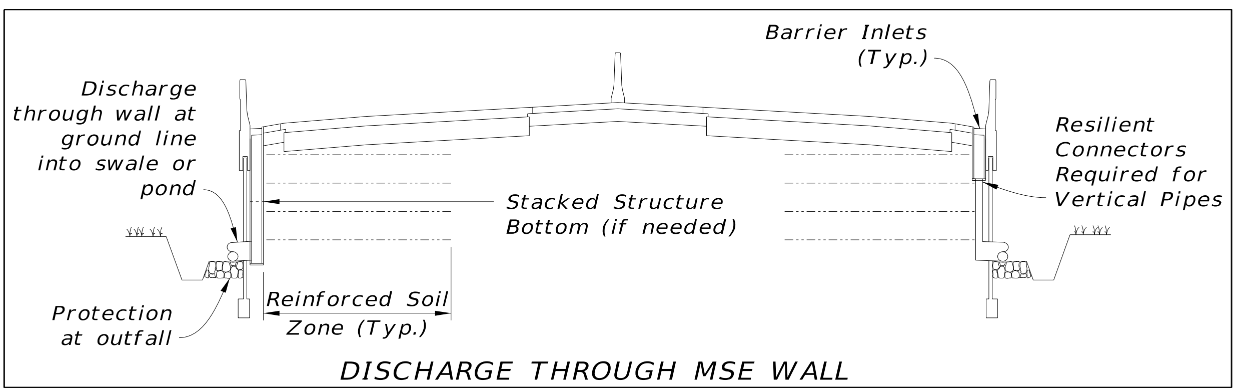


Figure 4-1

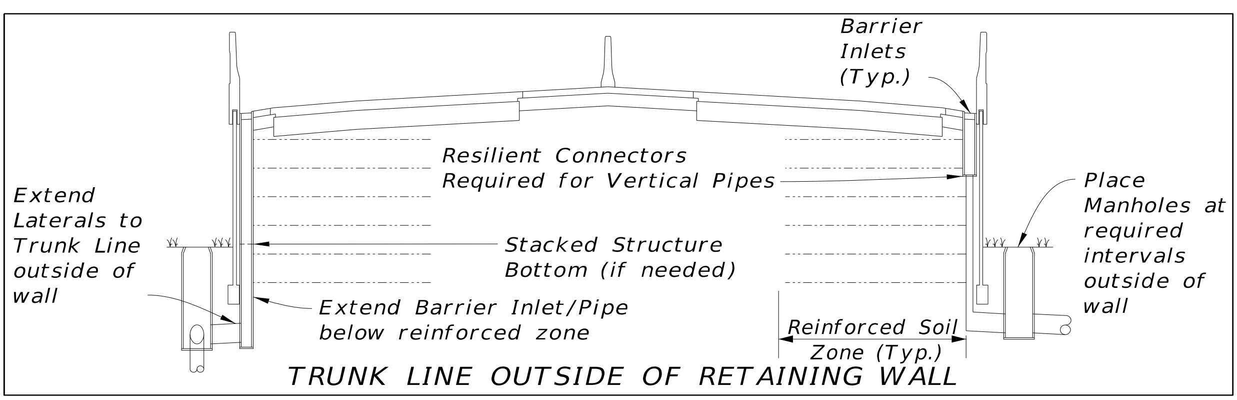


Figure 4-2

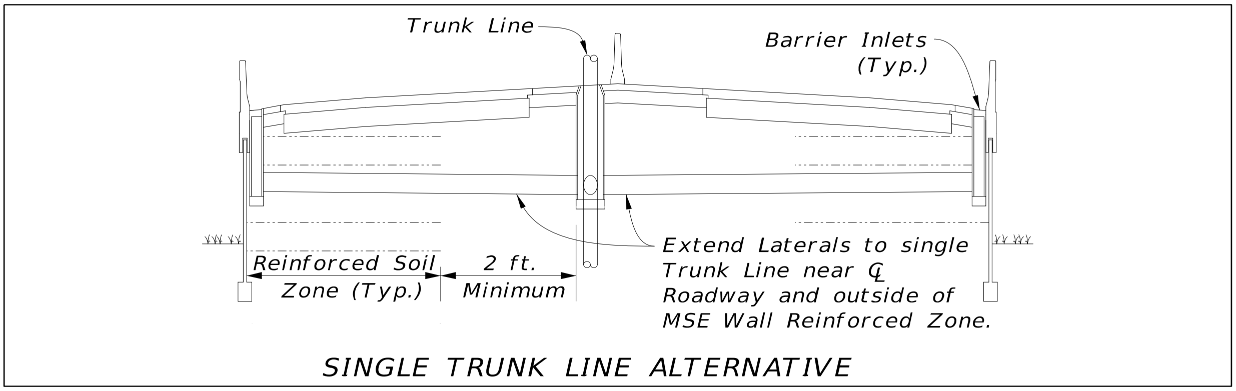


Figure 4-3

4.2 Profile Placement

4.2.1 Slopes

The drainage manual states that the minimum physical slope shall be that which will produce a velocity of 2.5 feet per second flowing full. The slope is obtained from the velocity form of Manning's equation using the full cross sectional area of the pipe:

$$V = (1.49 / n)\, R^{2/3}\, S^{1/2}$$

rearranging: $S = [V n / (1.49 R^{2/3})]^2$

where: R is based on full cross sectional area.
V = 2.5 fps

Table 4-1 provides the minimum physical slope for various pipe sizes with Manning's roughness coefficient of 0.012.

Minimum Physical Slope n = 0.012	
Diameter (inches)	Slope (%)
18	0.150
24	0.102
30	0.076
36	0.059
42	0.048
48	0.041
54	0.035
60	0.030
66	0.027
72	0.024
78	0.021
84	0.019

Table 4-1

For very flat systems, the minimum physical slope may not be realistic. The overall fall across the system is based on outlet pipe depth and structural clearances at the upper end. Most District Drainage Engineers will approve deviation from the minimum pipe slope in these cases.

Where the minimum slope cannot be attained, try to design the system to avoid appreciable drops in the velocity. This will help to carry sediment through the system instead of dropping sediment at some point in the system.

Please note that the design velocity is the actual velocity the pipe experiences under design conditions, not necessarily a theoretical Manning's velocity created by the physical slope of the pipe. Thus, when the design HGL is above the inside crown of the pipe (i.e. flowing full), the design velocity is the actual HGL velocity or the design flow divided by the pipe area. A minimum 0.1% physical trunk line pipe slope is recommended, but a steeper slope should be used wherever possible without causing overly deep cuts. Try to avoid a depth of cut that may result in the use of sheet pile. Usually, laterals can use a steeper slope unless utilities are in conflict. Install structure sumps if siltation is expected.

Related to the slope is the setting of flow lines. Refer to the Plans Preparation Manual, Volume II, Chapter 1 for accuracy that flow lines are to be displayed to.

4.2.2 Minimum Pipe Depth

The minimum depth of the pipe is controlled by either the minimum pipe cover or the need to have a clearance above the top of the pipe to maintain strength in a precast structure. Minimum pipe cover requirements are given in Appendix E of the Drainage Manual.

The loads placed on precast structures during shipping and handling are often greater than the loads placed on them in their final location. Since precast drainage structures are preferred by contractors and have become the industry standard, you should consider the potential for breakage during shipping and handling.

Where pipes are placed high in a structure, the structure has little if any, strength above the pipe. This can result in breakage during shipping or handling. For strength reasons, it is best to maintain a minimum amount of precast concrete section above the pipe.

The desirable amount of precast section varies with the type of inlet and bottom configuration. Generally, where a pipe is placed in grated inlets or in structure bottoms, try to maintain above the pipe opening a 6" precast section that has full wall thickness as shown in Figure 4-4. For ditch bottom inlets placed on J bottoms, the recommended minimum precast riser section varies depending if the unit has slots. Refer to Structure / Pipe configuration numbers 4 & 5 in Figure 4-8 (end of chapter). For ditch bottom inlets without slots, maintain a 10-inch riser section below the grate seat. For ditch bottom inlets with slots, maintain a 12-inch riser section below the slot.

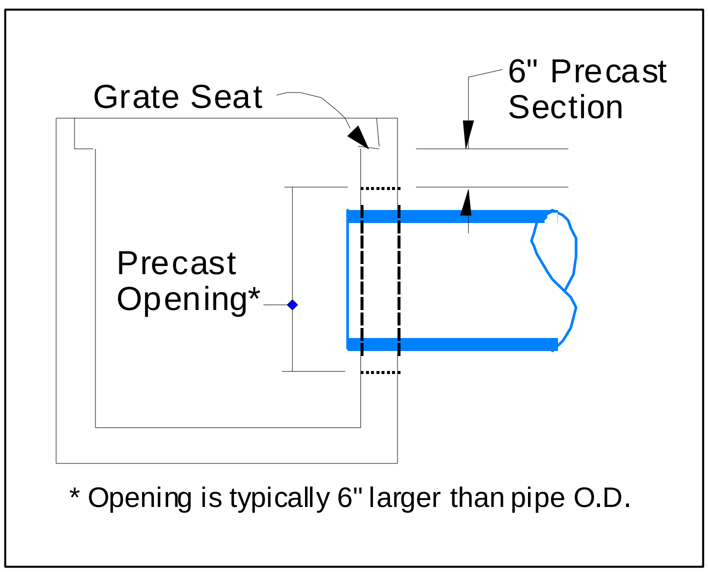


Figure 4-4

Tables 4-4 and 4-5 (end of chapter) give recommended minimum distances from inlets to pipe flow lines for most of the Department's standard inlets. These distances provide the precast section discussed above and are based on concrete pipe that is centered in the precast opening. The above discussion represents desirable values that you should try to achieve. On occasions it will be necessary to use less precast section than discussed above. This is acceptable because the contractor has the option to cast structures in place. Where using less, you must add all the appropriate dimensions, to assure that no conflict will exist between pipes and the structure.

4.3 Utility Coordination

During the design process, avoid utilities where practical without substantially increasing the cost of the storm drain system. Try to obtain information not only on the location of existing facilities, but proposed locations as well. The utility companies (both private and public) will view the design proposed on the Phase II plans as part of the utility coordination process. You may be asked to attend utility coordination meetings. These meetings can be very beneficial to the design effort because the concerned parties will be gathered together to resolve utility placement conflicts and the utility companies are accustomed to meeting face-to-face with FDOT representatives. The final storm drain design and utility locations are usually negotiated between the Department and the utility companies, with the goal to minimize the costs to the public. Sometimes minor changes in the storm drain design can reduce the cost to a utility company and minimize the cost to the public. At other times it may not be practical or cost effective to accommodate a utility company proposal. Utility companies often take the opportunity to upgrade their systems or add facilities during the Department's construction project. Do not assume they will relocate their systems in the process.

On projects with long storm drain systems in areas of many utilities, include one additional manhole in the quantities for unforeseen utility conflicts.

4.4 Pipe to Structure Connections

- When a bridge deck piping system connects to a roadway structure, a resilient connector should be used to accommodate the expected thermal movement of the bridge and its piping system.

- Check sizes of structure bottoms to make sure that the pipes fit. When doing so, use the outside diameter of concrete pipe[3]. It has the thickest wall of any of the optional pipe materials. Type P structure bottoms are either 4'-0" or smaller

---

3 - An easy way to remember the wall thickness of the concrete pipe is to take the inside diameter in feet and add one (1). The result is the wall thickness in inches. Examples: 30" pipe, I.D. = 2.5', Wall Thickness = 2.5 + 1 = 3.5".

diameter round (Alt A) or 3'-6" square (Alt B). 30" pipe is the maximum size to fit Type P bottoms. The contractor has the option of using either alternate A or B for Type P bottoms unless restricted by the plans. Type J structure bottoms are larger than Type P bottoms and come in various sizes as described in Index 200. The alternate and the size of the J bottoms are usually specified in the plans. Index 200 gives the minimal structure dimensions for various pipe sizes. Table 4-2 is an excerpt of Index 200 of the 2006 Design Standards. Refer to the latest version of the index for updates.

TABLE 3 - MINIMUM STRUCTURE SIZES FOR SINGLE PIPE CONNECTION PER SIDE

PIPE SIZE	RECTANGULAR		ROUND	
	Side Dimension (L)		Diameter (D)	
	Single Pipe Per Side	Note Number	Single Pipe or θ=180°	2 to 4 Pipes θ=90°
18"	3'-6"		3'-6"	4'-0"
24"	3'-6"		3'-6"	5'-0"
30"	3'-6" / 4'-0"	2	4'-0"	6'-0"
36"	4'-0" / 5'-0"	3	5'-0"	7'-0"
42"	5'-0"		6'-0"	7'-0"
48"	6'-0"		6'-0"	8'-0"
54"	6'-0"		7'-0"	10'-0"
60"	7'-0"		7'-0"	10'-0"
66"	7'-0" / 8'-0"	4	8'-0"	12'-0"
72"	8'-0"		8'-0"	12'-0"
78"	9'-0"		10'-0"	12'-0"
84"	9'-0"		12'-0"	N/A

TABLE 3 NOTES:

1. For Round Structures sizes with variable angles between pipes and variable pipe sizes, refer to the FDOT Storm Drain Handbook.
2. For 3'-6" Precast Square Structure Bottoms, 30" Pipes with similar invert elevations are not permitted in adjacent walls. Use 4'-0" Side Dimensions when 30" pipe openings are required on adjacent walls and the difference in flow lines is less than 3'-0".
3. For 4'-0" Precast Square Structure Bottoms, 36" Pipes with similar invert elevations are not permitted in adjacent walls. Use 5'-0" Side Dimensions when 36" pipe openings are required on adjacent walls and the difference in flow lines is less than 3'-0".
4. For 7'-0" Precast Square Structure Bottoms, 66" Pipes with similar invert elevations are not permitted in adjacent walls. Use 8'-0" Side Dimensions when 66" pipe openings are required on adjacent walls and the difference in flow lines is less than 4'-0".

Table 4-2: Excerpt of Index 200 of the 2006 Design Standards

- The skew that a pipe enters a precast rectangular structure is limited by the precast pipe opening. The maximum opening is 6 inches larger than the pipe outside diameter (Index 201). The maximum pipe skew varies with the structure wall thickness and the pipe size. The maximum skew for various pipe sizes passing through 8" structure walls is shown in Table 4-3. Index 200 provides skew values for 6" structure walls and other pipe sizes. Use round structure bottoms (Alternate A) where the pipe enters the structure at a larger angle.

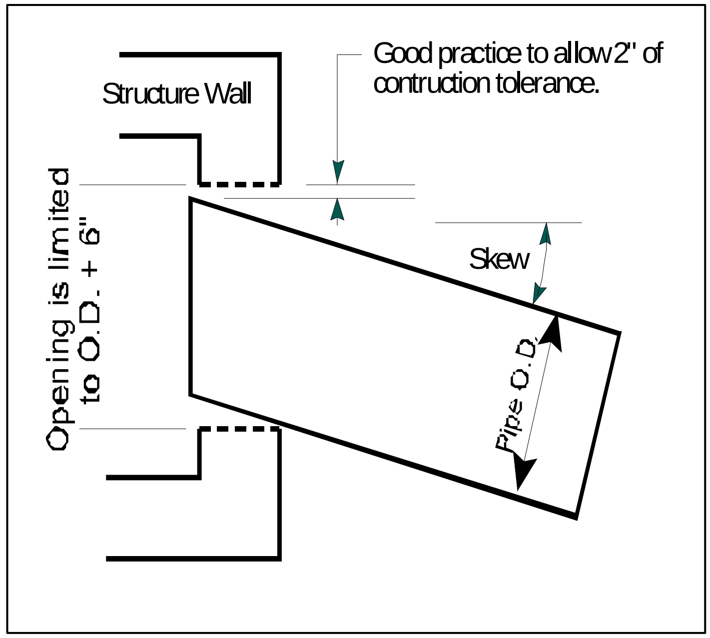


Figure 4-5

Index 201 includes a detail of a pipe opening at a corner of a structure. Although a detail exists for this condition, its use should be restricted to situations where other alternatives do not exist. The designer should make every attempt to ensure pipes do not enter the corner of rectangular structures (“corner-cutouts”).

Where placing pipes in existing rectangular structures, the maximum skew is limited by the dimension of the skewed pipe cut fitting between the walls.

	Pipe Size							
	18"	24"	30"	36"	42"	48"	54"	60"
Max. Skew	19^o	17^o	16^o	16^o	15^o	14^o	14^o	13^o

These values are based on 2" of construction tolerance, precast structures with 8" walls, and concrete pipe dimensions.

Table 4-3

- Where round structure bottoms are used consider the need to maintain a precast section between the openings of adjacent pipes. Try to maintain at least a 2" section along the inside wall between adjacent pipe openings as shown to the right. Table 4-6 (end of chapter) provides the minimum angle between adjacent pipe centerlines to maintain the 2" precast section along the inside wall. The values in Table 4-6 are based on equal pipe centerline elevations and standard concrete pipe openings. Using these minimum angles for pipes with offset centerline elevations and other pipe materials is conservative and would yield more than 2" of precast section.

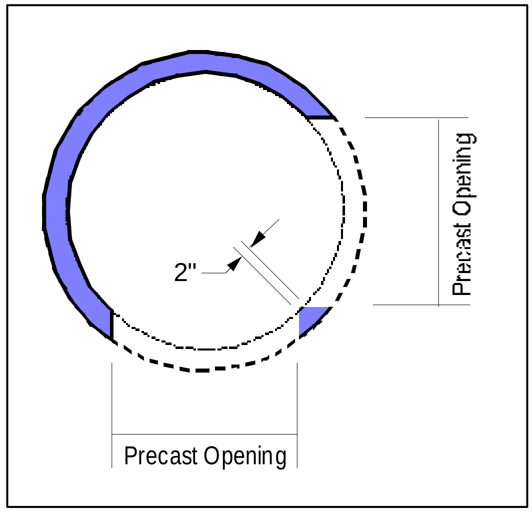


Figure 4-6

- Where large pipes are stubbed into the main line or a large main line pipe makes a 90^{o} turn, rectangular structures can be smaller than round structures given the same pipe sizes. Figure 4-7 shows 48" pipes making a 90^{o} turn at a structure. An 8' round structure is needed, while a 6' rectangular structure would work.

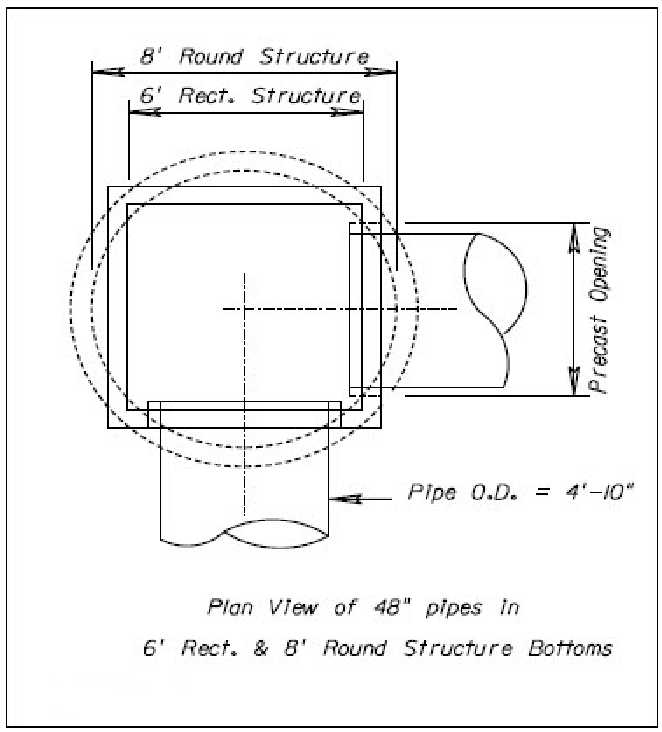


Figure 4-7

INLET TYPE	SLOT TYPE	PIPE LOCATION Wall		15" Pipe	18" Pipe	24" Pipe	30" Pipe	36" Pipe	42" Pipe	48" Pipe	54" Pipe	60" Pipe
				RECOMMENDED MIN. DISTANCE (Ft.) FROM GRATE (INLET) ELEVATION TO PIPE FLOW LINE								
Type A		Short	2'-0"	2.2 (1)	2.5 (1)	4.8 (5)	5.4 (5)	5.9 (5)	6.5 (5)	7.0 (5)	7.5 (5)	8.1 (5)
		Long	3'-1"	2.5 (2)	2.8 (2)	4.8 (5)	5.4 (5)	5.9 (5)	6.5 (5)	7.0 (5)	7.5 (5)	8.1 (5)
Type B (Note 3)	Travers	Short	No Slot	2.6 (2)	2.9 (2)	3.4 (2)	4.0 (2)	6.9 (4)	7.5 (4)	8.0 (4)	8.5 (4)	9.1 (4)
			Under Slot	3.4 (3)	3.6 (3)	4.2 (3)	4.7 (3)	6.9 (4)	7.5 (4)	8.0 (4)	8.5 (4)	9.1 (4)
		Long		2.6 (2)	2.9 (2)	3.4 (2)	4.0 (2)	4.5 (2)	7.5 (4)	8.0 (4)	8.5 (4)	9.1 (4)
Type C (Note 3)	None	Short	2'-0"	2.2 (1)	2.5 (1)	4.7 (5)	5.2 (5)	5.8 (5)	6.3 (5)	6.8 (5)	7.4 (5)	7.9 (5)
		Long	3'-1"	2.4 (2)	2.6 (2)	4.7 (5)	5.2 (5)	5.8 (5)	6.3 (5)	6.8 (5)	7.4 (5)	7.9 (5)
	Travers	Short	No Slot	2.2 (1)	2.5 (1)	5.3 (4)	5.8 (4)	6.3 (4)	6.9 (4)	7.4 (4)	8.0 (4)	8.5 (4)
			Under Slot	2.8 (3)	3.0 (3)	5.3 (4)	5.8 (4)	6.3 (4)	6.9 (4)	7.4 (4)	8.0 (4)	8.5 (4)
		Long		2.4 (2)	2.6 (2)	5.3 (4)	5.8 (4)	6.3 (4)	6.9 (4)	7.4 (4)	8.0 (4)	8.5 (4)
	Non-Trav 12" Std.	Short	No Slot	2.2 (1)	2.5 (1)	5.7 (4)	6.2 (4)	6.8 (4)	7.3 (4)	7.8 (4)	8.4 (4)	8.9 (4)
			Under Slot	3.2 (3)	3.5 (3)	5.7 (4)	6.2 (4)	6.8 (4)	7.3 (4)	7.8 (4)	8.4 (4)	8.9 (4)
		Long		2.4 (2)	2.6 (2)	5.7 (4)	6.2 (4)	6.8 (4)	7.3 (4)	7.8 (4)	8.4 (4)	8.9 (4)
Type D (Note 3)	None	Short	3-1"	2.4 (2)	2.6 (2)	3.2 (2)	5.2 (5)	5.8 (5)	6.3 (5)	6.8 (5)	7.4 (5)	7.9 (5)
		Long	4'-1"	2.2 (1)	2.5 (1)	3.0 (1)	3.5 (1)	4.1 (1)	6.3 (5)	6.8 (5)	7.4 (5)	7.9 (5)
	Travers	Short		2.4 (2)	2.6 (2)	3.2 (2)	5.8 (4)	6.3 (4)	6.9 (4)	7.4 (4)	8.0 (4)	8.5 (4)
		Long	No Slot	2.2 (1)	2.5 (1)	3.0 (1)	3.5 (1)	4.1 (1)	6.9 (4)	7.4 (4)	8.0 (4)	8.5 (4)
			Under Slot	2.8 (3)	3.0 (3)	3.6 (3)	4.1 (3)	4.7 (3)	6.9 (4)	7.4 (4)	8.0 (4)	8.5 (4)
	Non-Trav 12" Std.	Short		2.4 (2)	2.6 (2)	3.2 (2)	6.2 (4)	6.8 (4)	7.3 (4)	7.8 (4)	8.4 (4)	8.9 (4)
		Long	No Slot	2.2 (1)	2.5 (1)	3.0 (1)	3.5 (1)	4.1 (1)	7.3 (4)	7.8 (4)	8.4 (4)	8.9 (4)
			Under Slot	3.2 (3)	3.5 (3)	4.0 (3)	4.5 (3)	5.1 (3)	7.3 (4)	7.8 (4)	8.4 (4)	8.9 (4)
Type E (Note 3)	None	Short	3'-0"	2.2 (1)	2.5 (1)	3.0 (1)	5.2 (5)	5.8 (5)	6.3 (5)	6.8 (5)	7.4 (5)	7.9 (5)
		Long	4'-6"	2.4 (2)	2.6 (2)	3.2 (2)	3.7 (2)	4.3 (2)	6.3 (5)	6.8 (5)	7.4 (5)	7.9 (5)
	Travers	Short	No Slot	2.2 (1)	2.5 (1)	3.0 (1)	5.8 (4)	6.3 (4)	6.9 (4)	7.4 (4)	8.0 (4)	8.5 (4)
			Under Slot	2.8 (3)	3.0 (3)	3.6 (3)	5.8 (4)	6.3 (4)	6.9 (4)	7.4 (4)	8.0 (4)	8.5 (4)
		Long		2.4 (2)	2.6 (2)	3.2 (2)	3.7 (2)	4.3 (2)	6.9 (4)	7.4 (4)	8.0 (4)	8.5 (4)
	Non-Trav 12" Std.	Short	No Slot	2.2 (1)	2.5 (1)	3.0 (1)	6.2 (4)	6.8 (4)	7.3 (4)	7.8 (4)	8.4 (4)	8.9 (4)
			Under Slot	3.2 (3)	3.5 (3)	4.0 (3)	6.2 (4)	6.8 (4)	7.3 (4)	7.8 (4)	8.4 (4)	8.9 (4)
		Long		2.4 (2)	2.6 (2)	3.2 (2)	3.7 (2)	4.3 (2)	7.3 (4)	7.8 (4)	8.4 (4)	8.9 (4)

Notes: 1. The number in parentheses ( ) refers to one of the structure pipe combinations shown in Figure 4-4.

2. ***** CAUTION ***** Where multiple pipes enter a structure, needing a J-bottom because of one pipe could dictate greater distances than shown above for other pipes entering the structure.

3. The values shown for Type B, C, D, & E inlets are based on Alt. B Bottoms. Alternate A Bottoms have thicker slabs, so add 2 inches for up through 6' diameter bottoms. Add 4 inches for 8' diameter bottoms.

4. The distances are based on precast structures and standard precast openings for concrete pipes.

5. The designer should check that the minimum cover requirements of Drainage Manual Appendix E are met.

TABLE 4-4

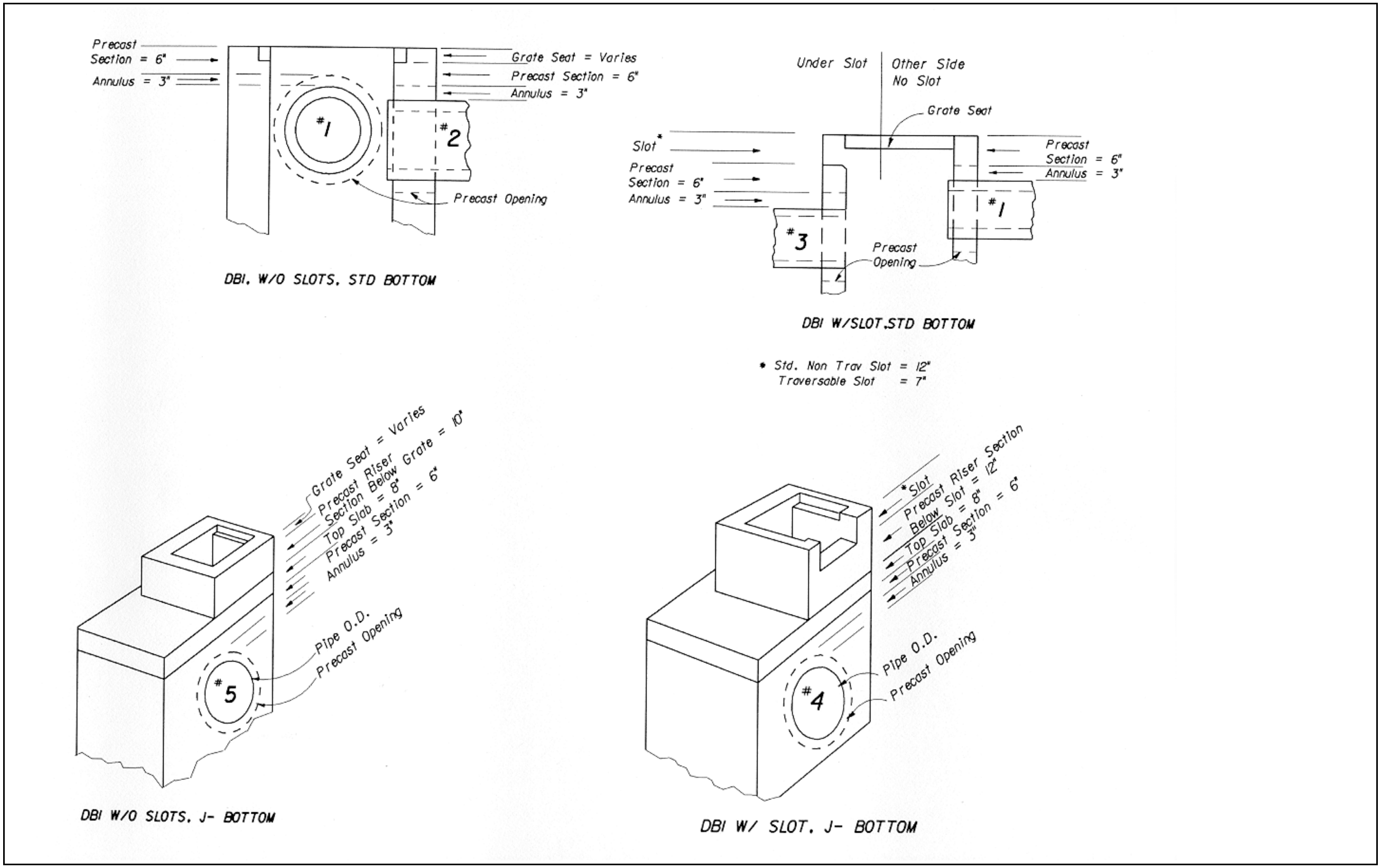

Precast Section = 6"
Annulus = 3"
Grate Seat = Varies
Precast Section = 6"
Annulus = 3"
#1
#2
Precast Opening
DBI. W/O SLOTS, STD BOTTOM
Under Slot
Other Side No Slot
Grate Seat
Slot*
Precast Section = 6"
Annulus = 3"
Precast Section = 6"
Annulus = 3"
#3
#1
Precast Opening
DBI W/SLOT.STD BOTTOM
* Std. Non Trav Slot = 12"
Traversable Slot = 7"
Grate Seat = Varies
Precast Riser Section Below Grate = 10"
Top Slab = 8"
Precast Section = 6"
Annulus = 3"
Pipe O.D.
Precast Opening
#5
DBI W/O SLOTS, J- BOTTOM
*Slot
Precast Riser Section Below Slot = 12"
Top Slab = 8"
Precast Section = 6"
Annulus = 3"
Pipe O.D.
Precast Opening
#4
DBI W/ SLOT, J- BOTTOM

Figure 4-8

INLET TYPE	SLOT TYPE	PIPE LOCATION		RECOMMENDED MIN. DISTANCE (Ft.) FROM GRATE (INLET) ELEVATION TO PIPE FLOW LINE								
		Wall	Wall Dim.	15" Pipe	18" Pipe	24" Pipe	30" Pipe	36" Pipe	42" Pipe	48" Pipe	54" Pipe	60" Pipe
Type F	n/a	Short	2'-6"	2.2 (1)	2.5 (1)	4.8 (5)	5.3 (5)	5.8 (5)	6.4 (5)	6.9 (5)	7.5 (5)	8.0 (5)
		Long	4'-0"	2.4 (2)	2.7 (2)	3.3 (2)	3.8 (2)	4.3 (2)	6.4 (5)	6.9 (5)	7.5 (5)	8.0 (5)
Type H	None	Short	3'-0"	2.2 (1)	2.5 (1)	3.0 (1)	n/a	n/a	n/a	n/a	n/a	n/a
		Long	6'-7"	2.4 (2)	2.6 (2)	3.2 (2)	3.7 (2)	4.3 (2)	4.8 (2)	5.3 (2)	5.9 (2)	6.4 (2)
	Non-Trav 12" std.	Short	3'-0"	3.2 (3)	3.5 (3)	4.0 (3)	n/a	n/a	n/a	n/a	n/a	n/a
		Long	6'-7"	2.4 (2)	2.6 (2)	3.2 (2)	3.7 (2)	4.3 (2)	4.8 (2)	5.3 (2)	5.9 (2)	6.4 (2)
Type J	n/a	Short	3'-3"	2.6 (2)	2.9 (2)	3.4 (2)	5.5 (5)	6.0 (5)	6.5 (5)	7.1 (5)	7.6 (5)	8.2 (5)
		Long	3'-10"	2.4 (2)	2.7 (2)	3.3 (2)	3.8 (2)	6.0 (5)	6.5 (5)	7.1 (5)	7.6 (5)	8.2 (5)
Type S	n/a	Short	3'-3"	2.6 (2)	2.9 (2)	3.5 (2)	5.5 (5)	6.0 (5)	6.6 (5)	7.1 (5)	7.7 (5)	8.2 (5)
		Long	3'-10"	2.3 (2)	2.5 (2)	3.1 (2)	3.6 (2)	6.0 (5)	6.6 (5)	7.1 (5)	7.7 (5)	8.2 (5)
Type V	n/a	Short	3'-3"	2.6 (2)	2.9 (2)	3.4 (2)	5.5 (5)	6.0 (5)	6.5 (5)	7.1 (5)	7.6 (5)	8.2 (5)
		Long	3'-10"	2.4 (2)	2.7 (2)	3.3 (2)	3.8 (2)	6.0 (5)	6.5 (5)	7.1 (5)	7.6 (5)	8.2 (5)
Manhole Type 8	n/a	n/a		**RECOMMENDED MIN. DISTANCE (Ft.) FROM TOP ELEVATION TO PIPE FLOW LINE**								
				3.7 (10)	4.0 (10)	4.5 (10)	5.0 (10)	6.3 (11)	6.8 (11)	7.3 (11)	7.9 (11)	8.4 (11)
Barr-Wall 218	n/a			**RECOMMENDED MIN. DISTANCE (Ft.) FROM LOW POINT OF GRATE TO PIPE FLOW LINE**								
		Short	3'-3"	4.2 (8)	4.5 (8)	5.0 (8)	6.2 (9)	6.8 (9)	7.3 (9)	7.8 (9)	8.4 (9)	8.9 (9)
		Long	3'-8"	4.2 (8)	4.5 (8)	5.0 (8)	5.5 (8)	6.8 (9)	7.3 (9)	7.8 (9)	8.4 (9)	8.9 (9)
Curb 1-9	n/a	n/a		**RECOMMENDED MIN. DISTANCE (Ft.) FROM EDGE OF PAVEMENT TO PIPE FLOW LINE**								
				3.9 (6)	4.2 (6)	4.7 (6)	5.3 (6)	6.5 (7)	7.0 (7)	7.5 (7)	8.1 (7)	8.6 (7)

Notes: 1. The number in parentheses ( ) refers to one of the structure pipe combinations shown in Figure 4-4 and 4-5.

2. ***** CAUTION ***** Where multiple pipes enter a structure, needing a J-bottom because of one pipe could dictate greater distances than shown above for other pipes entering the structure.

3. ***** CAUTION ***** For curb inlets and manholes, where 30" pipes with similar inverts enter a structure at 90 degrees, a J-bottom is required, thus the minimum distance may be greater than shown above. This may apply to other inlets also.

4. The distances are based on precast structures and standard precast openings for concrete pipes.

5. The designer should check that the minimum cover requirements of Drainage Manual Appendix E are met.

TABLE 4-5

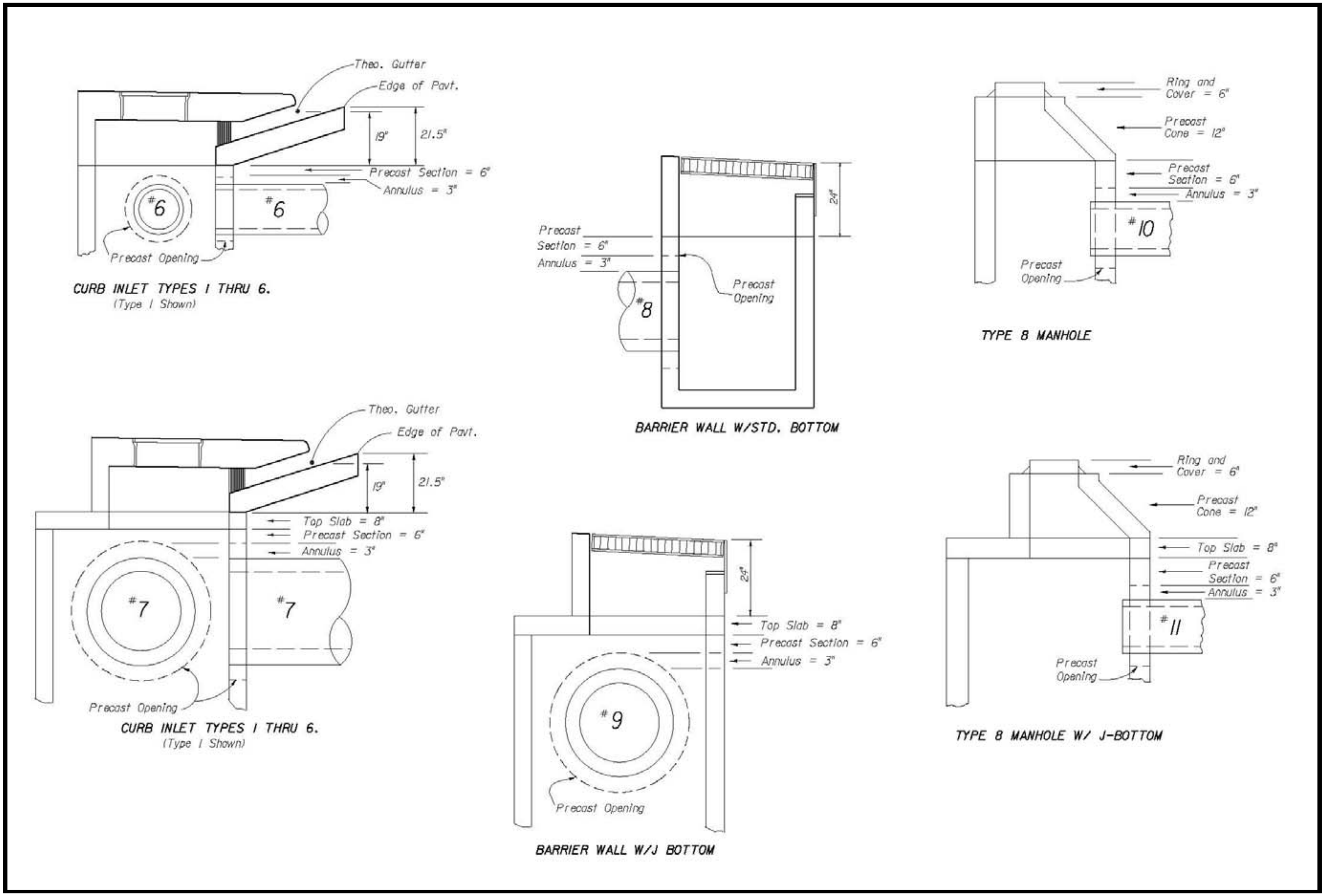

Theo. Gutter
Edge of Pavt.
19"
21.5"
Precast Section = 6"
Annulus = 3"
#6
#6
Precast Opening
CURB INLET TYPES I THRU 6.
(Type I Shown)
24"
Precast Section = 6"
Annulus = 3"
Precast Opening
#8
BARRIER WALL W/STD. BOTTOM
Ring and Cover = 6"
Precast Cone = 12"
Precast Section = 6"
Annulus = 3"
#10
Precast Opening
TYPE 8 MANHOLE
Theo. Gutter
Edge of Pavt.
19"
21.5"
Top Slab = 8"
Precast Section = 6"
Annulus = 3"
#7
#7
Precast Opening
CURB INLET TYPES I THRU 6.
(Type I Shown)
24"
Top Slab = 8"
Precast Section = 6"
Annulus = 3"
#9
Precast Opening
BARRIER WALL W/J BOTTOM
Ring and Cover = 6"
Precast Cone = 12"
Top Slab = 8"
Precast Section = 6"
Annulus = 3"
#11
Precast Opening
TYPE 8 MANHOLE W/ J-BOTTOM

Figure 4-9

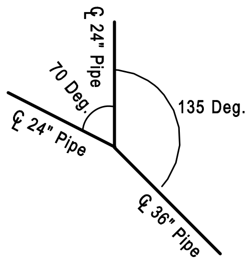
RECOMMENDED MINIMUM ANGLE (in Degrees) BETWEEN ADJACENT PIPE CENTER LINES IN ROUND (Alt. A) STRUCTURE BOTTOMS

PIPE SIZE	ADJACENT PIPE SIZE																			
	18"		24"		30"		36"		42"		48"		54"		60"		66"		72"	
18"	26	*12'*	28	*12'*	31	*12'*	33	*12'*	38	*12'*	41	*12'*	44	*12'*	46	*12'*	49	*12'*	52	*12'*
	31	*10'*	34	*10'*	37	*10'*	40	*10'*	46	*10'*	50	*10'*	53	*10'*	57	*10'*	61	*10'*	65	*10'*
	39	*8'*	43	*8'*	47	*8'*	51	*8'*	59	*8'*	64	*8'*	69	*8'*	75	*8'*	82	*8'*	90	*8'*
	45	*7'*	49	*7'*	54	*7'*	59	*7'*	69	*7'*	75	*7'*	83	*7'*	92	*7'*	114	*7'*		
	52	*6'*	58	*6'*	63	*6'*	70	*6'*	84	*6'*	94	*6'*								
	64	*5'*	71	*5'*	78	*5'*	87	*5'*												
	82	*4'*	90	*4'*	105	*4'*														
24"			31	*12'*	33	*12'*	36	*12'*	41	*12'*	43	*12'*	46	*12'*	49	*12'*	52	*12'*	55	*12'*
			37	*10'*	40	*10'*	43	*10'*	49	*10'*	53	*10'*	56	*10'*	60	*10'*	64	*10'*	68	*10'*
			46	*8'*	50	*8'*	54	*8'*	63	*8'*	68	*8'*	73	*8'*	79	*8'*	85	*8'*	94	*8'*
			53	*7'*	58	*7'*	63	*7'*	74	*7'*	80	*7'*	87	*7'*	96	*7'*	118	*7'*		
			63	*6'*	69	*6'*	75	*6'*	90	*6'*	100	*6'*								
			78	*5'*	85	*5'*	94	*5'*												
			102	*4'*	114	*4'*														
30"					36	*12'*	38	*12'*	43	*12'*	46	*12'*	49	*12'*	51	*12'*	54	*12'*	57	*12'*
					43	*10'*	46	*10'*	52	*10'*	56	*10'*	59	*10'*	63	*10'*	67	*10'*	71	*10'*
					54	*8'*	58	*8'*	67	*8'*	72	*8'*	77	*8'*	83	*8'*	89	*8'*	98	*8'*
					63	*7'*	68	*7'*	78	*7'*	85	*7'*	92	*7'*	101	*7'*	123	*7'*		
					75	*6'*	81	*6'*	95	*6'*	105	*6'*								
					93	*5'*	101	*5'*												
					127	*4'*														
36"							41	*12'*	46	*12'*	48	*12'*	51	*12'*	54	*12'*	57	*12'*	60	*12'*
							49	*10'*	55	*10'*	59	*10'*	62	*10'*	66	*10'*	70	*10'*	74	*10'*
							62	*8'*	71	*8'*	76	*8'*	81	*8'*	87	*8'*	93	*8'*	102	*8'*
							72	*7'*	83	*7'*	89	*7'*	97	*7'*	106	*7'*	128	*7'*		
							91	*6'*	101	*6'*	111	*6'*								
							110	*5'*												
42"									51	*12'*	53	*12'*	56	*12'*	59	*12'*	62	*12'*	65	*12'*
									62	*10'*	65	*10'*	69	*10'*	72	*10'*	76	*10'*	81	*10'*
									80	*8'*	85	*8'*	90	*8'*	95	*8'*	102	*8'*	111	*8'*
									90	*7'*	100	*7'*	107	*7'*	117	*7'*	138	*7'*		
									116	*6'*	126	*6'*								
48"											56	*12'*	59	*12'*	62	*12'*	65	*12'*	68	*12'*
											69	*10'*	72	*10'*	76	*10'*	80	*10'*	84	*10'*
											89	*8'*	94	*8'*	100	*8'*	107	*8'*	115	*8'*
											106	*7'*	114	*7'*	123	*7'*	145	*7'*		
											136	*6'*								
54"													62	*12'*	64	*12'*	67	*12'*	70	*12'*
													76	*10'*	79	*10'*	83	*10'*	87	*10'*
													100	*8'*	105	*8'*	112	*8'*	121	*8'*
													121	*7'*	130	*7'*	152	*7'*		
60"															67	*12'*	70	*12'*	73	*12'*
															83	*10'*	87	*10'*	91	*10'*
															111	*8'*	118	*8'*	126	*8'*
															139	*7'*	161	*7'*		
66"																	73	*12'*	76	*12'*
																	91	*10'*	95	*10'*
																	124	*8'*	133	*8'*
72"																			79	*12'*
																			99	*10'*
																			142	*8'*

Notes: 1. The italicized numbers to the right of the degree values are the structure bottom diameters.

2. The values are based on the pipe center lines being at equal elevation, a 2" precast section along the inside structure wall between adjacent pipes, and standard precast openings for concrete pipe. The sizes of the precast openings are those proposed by the Florida Precast Concrete Structures Association and are not always O.D. plus 6".

3. The value for two 36" pipes in a 6' diameter structure is adjusted to be consistent with Index no. 200. A similar change was made for two 42" pipes in a 7' diameter structure.



Example:

What size round bottom should be used for these pipes?

1. Looking at the 2-24" adjacent pipes, the minimum internal angle is 78^{o} for a 5' dia. bottom and 63^{o} for a 6' dia. bottom. Since these pipes enter at 70^{o}, we need a 6' dia. bottom.

2. Checking the adjacent 24" and 36" pipes, 75^{o} is needed between the pipes in a 6' dia. bottom. We have 135^{o}, so the 6' dia. bottom works.

Table 4-6

Chapter 5 - Pipe Hydraulics

The Drainage Manual states that friction losses shall be considered in the computation of the design hydraulic gradient for all storm drain systems. Energy losses associated with pollution control structures (weirs and baffles) and utility conflict structures shall also be considered where present in a system. When the hydraulic calculations consider only the above, the elevation of the hydraulic gradient shall be at least 1 foot below the theoretical gutter elevation. This is equivalent to 13.5” (1.13’) below the edge of pavement for sections with Type E or F curb and gutter. For gutter inlets (Indexes 220 & 221), ditch bottom inlets (Indexes 230 through 235), and barrier wall inlets (Index 218) the 1 foot of clearance is applied to the grate elevation. For barrier wall inlets (Indexes 217 & 219) the 1 foot of clearance is applied to the theoretical grade point.

If all minor energy losses are calculated, it is acceptable for the hydraulic grade line to reach the theoretical gutter elevation. Minor losses include all the losses at inlets, manholes, and junctions, due to expansion, contraction, and changes in flow direction. Minor losses also include exit losses at the outlet of the system.

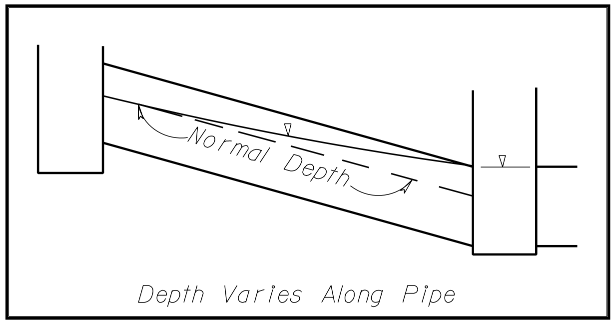
5.1 Pressure Flow

Under pressure flow conditions the pipe section flows full throughout. Friction losses are calculated using Manning’s equation with the flow area equal to the full cross sectional area of the pipe.

$$\text{Head loss [in feet]} = \frac{29n^2LV^2}{R^{1.33}\,2g} = \frac{4.61n^2LQ^2}{D^{5.33}}$$

Where: n = roughness coefficient (Refer to the Drainage Manual)
L = pipe length (feet)
V = velocity (fps)
Q = flow rate in (cfs)
R = hydraulic radius (feet) = Area / wetted perimeter
D = pipe diameter (feet)
g = gravitational constant = 32.2 ft/s^2

5.2 Partially Full Flow



For pipes that are flowing partially full, the calculations are more complicated. The cross-sectional flow area actually changes as the flow goes through the pipe. For example, the flow area at the downstream end of the pipe shown here is the full cross section area, but at the upstream end the flow area is much less.

The most accurate approach to calculating this is to do water surface profile calculations through the pipe section. Although acceptable, these calculations are tedious and not usually required. The Department accepts the following approach to calculating the hydraulics of partial full and pressure pipes.

Three values must be determined for each pipe section. These are the Lower End Hydraulic Gradient, the Upper End Hydraulic Gradient, and the flow velocity.

5.3 Lower End Hydraulic Gradient

Either the downstream hydraulic gradient or the flow conditions in the pipe controls the lower end HG. So you must often compare the water surface elevations associated with these and use the higher of the two as the Lower End HG.

> The downstream hydraulic gradient elevation is the downstream pipe Upper End HG elevation plus junction losses, if they are calculated.

Where the downstream HG is above the lower end crown of the subject pipe, the Lower End HG is the downstream HG. See Detail A of Figure 5-1. Pipe flow conditions will not control and comparing water surface elevations is not necessary. If the downstream HG is below the lower end crown, you will need to compare the downstream HG with the water elevation associated with the pipe flow conditions.

Where the downstream hydraulic gradient is low enough, one of two pipe flow conditions will control the Lower End HG. See Detail C & D of Figure 5-1. The appropriate flow condition is dependent on the relationship of the physical pipe slope and the full flow friction slope. If the pipe is sloped steeper than the full flow friction slope, it is reasonable to assume that normal depth flow exists at the lower end. Then the Lower End HG is the normal depth plus the lower end flow line elevation. (Actual depth could be above normal depth because the pipe was not long enough to allow normal depth to be reached).

Figures 5-5 and 5-6 (end of chapter), or the Department's hydraulic calculator can be used to get normal flow depth and associated velocity.

If the pipe slope is equal to or flatter than the full flow friction slope, the pipe is flowing full over most of its length. Although the flow may be dropping through critical depth[4] near the outlet, assuming full flow at the outlet is reasonable and conservative. During very low flow rates even flat pipes will not flow full, but such low rates are not typical for design conditions.

In short, use the higher of the following for the Lower End HG as shown in Figure 5-1.

Condition 1: The downstream pipe Upper End HG (+ junction losses, if calculated)

OR

Condition 2: The normal depth + Lower End flow line elevation (for pipes sloped steeper than full flow friction slope) or Lower End crown elevation (for pipes sloped equal to or flatter than full flow friction slope.

For the outlet pipe of the system, the Lower end HG elevation is the Design Tailwater elevation.

4 - For a slightly more refined analysis in this situation, midway between critical depth and the crown of the pipe of [(D_c+ D)/2} could be used as the Lower End HG.

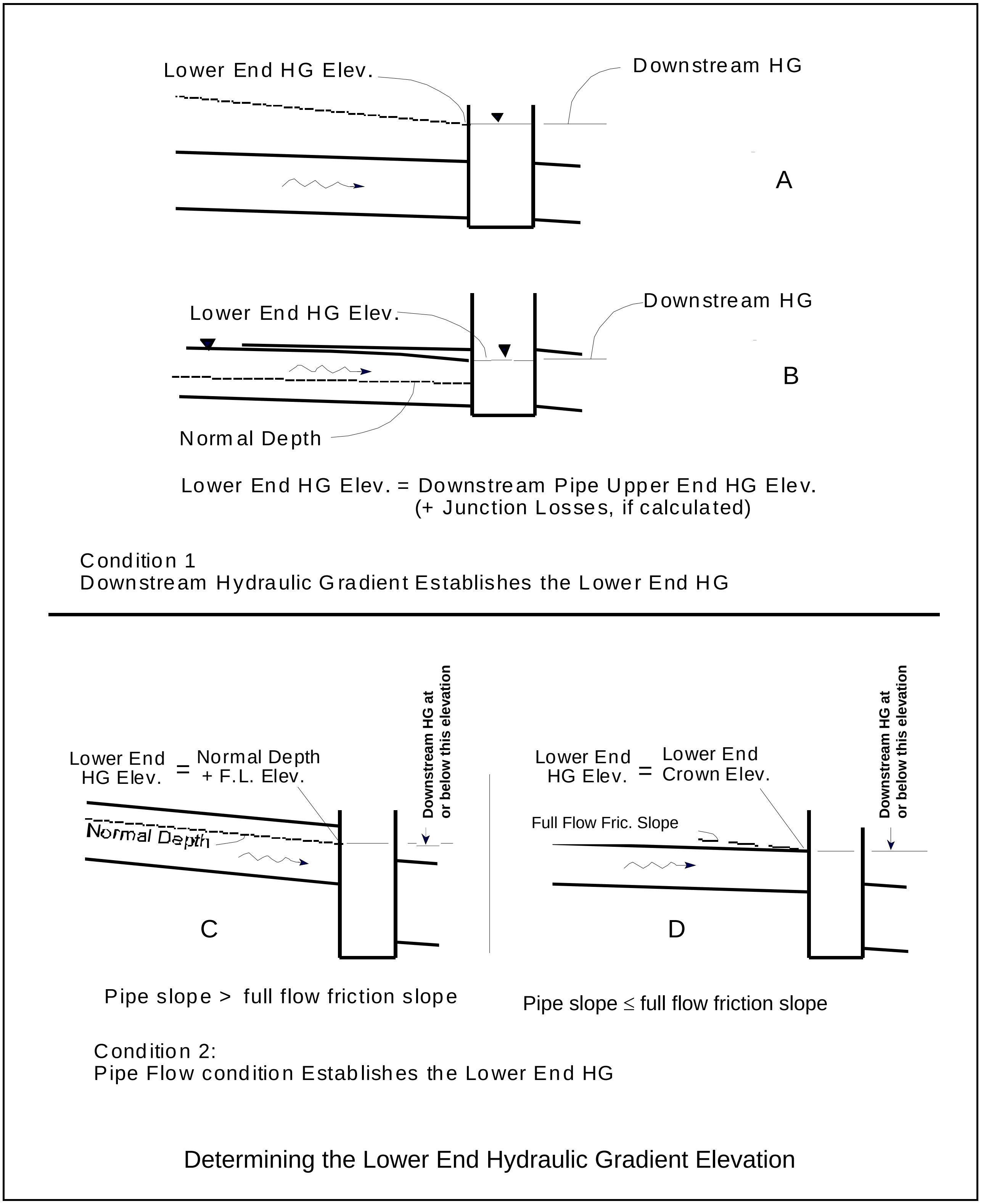

Lower End HG Elev.
Downstream HG
A
Lower End HG Elev.
Downstream HG
B
Normal Depth
Lower End HG Elev. = Downstream Pipe Upper End HG Elev.
(+ Junction Losses, if calculated)
Condition 1
Downstream Hydraulic Gradient Establishes the Lower End HG
Lower End HG Elev. = Normal Depth + F.L. Elev.
Downstream HG at or below this elevation
Normal Depth
C
Lower End HG Elev. = Lower End Crown Elev.
Full Flow Fric. Slope
Downstream HG at or below this elevation
D
Pipe slope > full flow friction slope
Pipe slope ≤ full flow friction slope
Condition 2:
Pipe Flow condition Establishes the Lower End HG
Determining the Lower End Hydraulic Gradient Elevation

Figure 5-1

5.3.1 Design Tailwater (DTW)

The Drainage Manual gives the standard Design Tailwater conditions. In general, it says use the higher of: crown of the pipe or the downstream condition. Stormwater ponds are the commonly constructed at the outlet of storm drains, so the pond stage may be the design tailwater. Some Districts may have more stringent criteria than shown in Drainage Manual.

The pond stage can be determined by “routing” the storm drain design event (frequency) through the pond. “Routing” refers to the use of the storage indication method that is commonly used to simulate runoff hydrographs flowing through stormwater management facilities. HEC-22 contains a discussion and example of the storage indication method.

5.4 Upper End Hydraulic Gradient

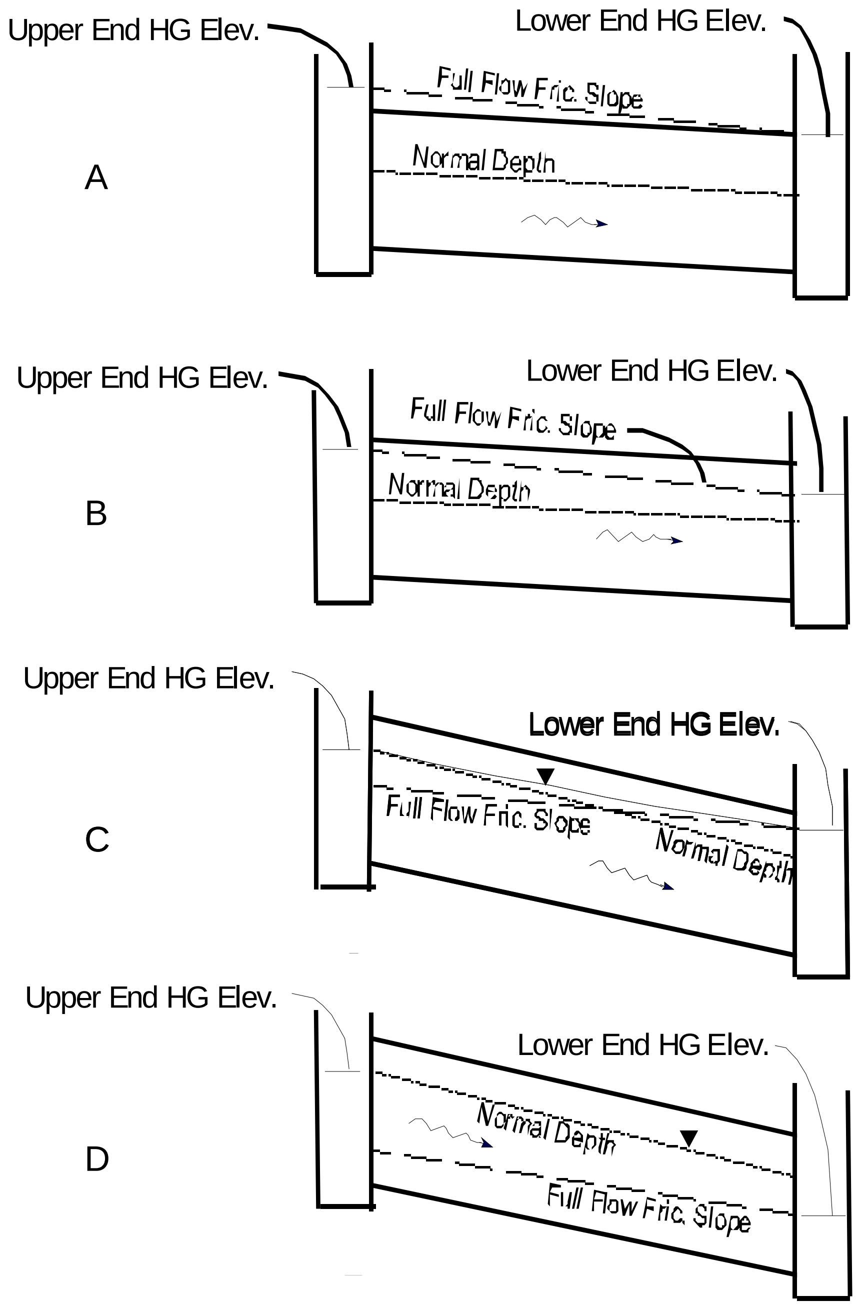
Use the higher of the following as shown in Figure 5-2:

Condition 1: The Lower End HG plus the full flow friction loss

OR

Condition 2: The elevation of normal depth in the pipe at the upper end.

A comparison may not be necessary. First add the full flow friction loss to the Lower End HG. If this is above the Upper End crown, there is no need to calculate normal depth. The Lower End HG plus full flow friction loss will control.



The Upper End HG is higher of the Lower End HG + Full Flow Friction loss (A & B) or Normal Depth + F.L. (C & D)

Determining the Upper End Hydraulic Elevation

Figure 5-2

5.5 Flow Velocity in the Pipe Section

For pressure flow pipes, the velocity is based on the full cross section area.

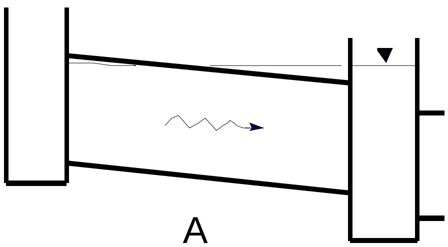
$$\text{Velocity(fps)} = \frac{Q}{A} = \frac{Q}{\frac{\pi D^2}{4}}$$

Where the tailwater conditions submerge the storm drain without stormwater flow, the travel time in the pipe can be ignored thus the velocity is irrelevant. See the discussion of ignoring travel time in Chapter 2.

Where: Q = Flow Rate (cfs)
D = Diameter (feet)

For pipes flowing partially full, it can be more complicated to determine the velocity. There can be a water surface profile in the pipe so the cross-sectional flow area can change, thus changing the velocity along the pipe section. The most accurate velocity should represent the average velocity through the pipe section. Assuming the velocity associated with normal depth is a conservative assumption. See Figure 5-3.

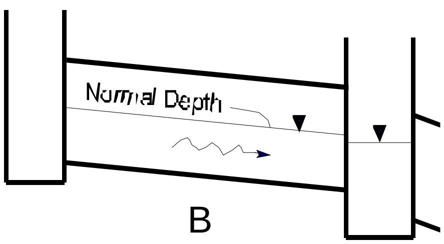
Figures 5-5 and 5-6, or the Department's hydraulic calculator can be used to get normal flow depth and associated velocity.



A

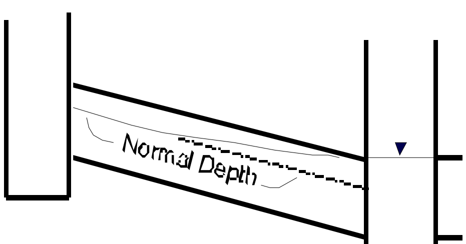
If the water surface in the downstream junction box substantially submerges the entire pipe(downstream water surface is near the crown of the upstream end) as shown here, it is reasonable to use the full flow velocity.

For any other partial flow conditions, use velocity associated with normal depth. This is reasonable for condition B and conservative for the other conditions.



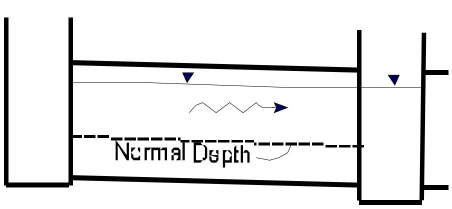
B

The water surface in the downstream junction box is at or below the normal depth.



C

The water surface in the downstream junction box submerges the outlet of the pipe, but would allow normal depth at the upstream end of the pipe.



D

Flat pipes with low flow where the depth is controlled by the water surface in the downstream junction box.

Determining the Velocity in Partially Filled Pipes

Figure 5-3

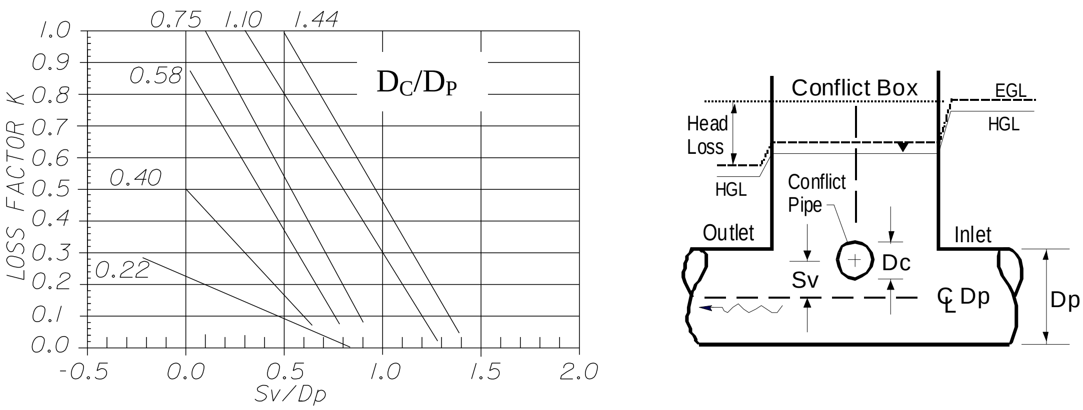
5.6 Utility Conflict Box Losses

Calculate the loss through a utility conflict box using the equation:

$$\text{Head Loss [in feet]} = K\frac{V^2}{2g}$$

Where: K = loss factor (or coefficient)
V = flow velocity in the storm drain (ft/s)
g = gravitational constant = 32.2 ft/s^2

Use Figure 5-4 to determine the loss factor in conflict boxes where the pipes are flowing full.



Notes:

1. The loss factors were obtained under full flow conditions, conflict centered between storm drain inlet and outlet.
2. Where two or more conflict pipes are closely spaced and one is above the other, treat the conflict as a single obstruction with an effective diameter equivalent to the sum of the two pipe diameters.
3. No correction factor is required for conflict pipes angled within the horizontal plane. Configurations were tested at a 45-degree angle.
4. This information is based on research by the University of South Florida and is documented in two reports. The first is "*Hydraulic Performance of Conflict Junction Boxes,*" July 1996; WPI no. 0510710; contract no. B-9080. The second is "*Hydraulic Performance of Conflict Manholes,*" November 1999; WPI no. 0510819; contract no. B-B304. Contact the FDOT Research Center at 850-414-4615 to obtain copies.

Figure 5-4: Loss Factors for Conflict Manholes

5.7 Minor Losses

Minor losses are all the losses that are not due to friction. Generally these are energy losses due to changes or disturbances in the flow path. They include such things as entrance, exit, bend and junction losses. The losses are calculated from the equation

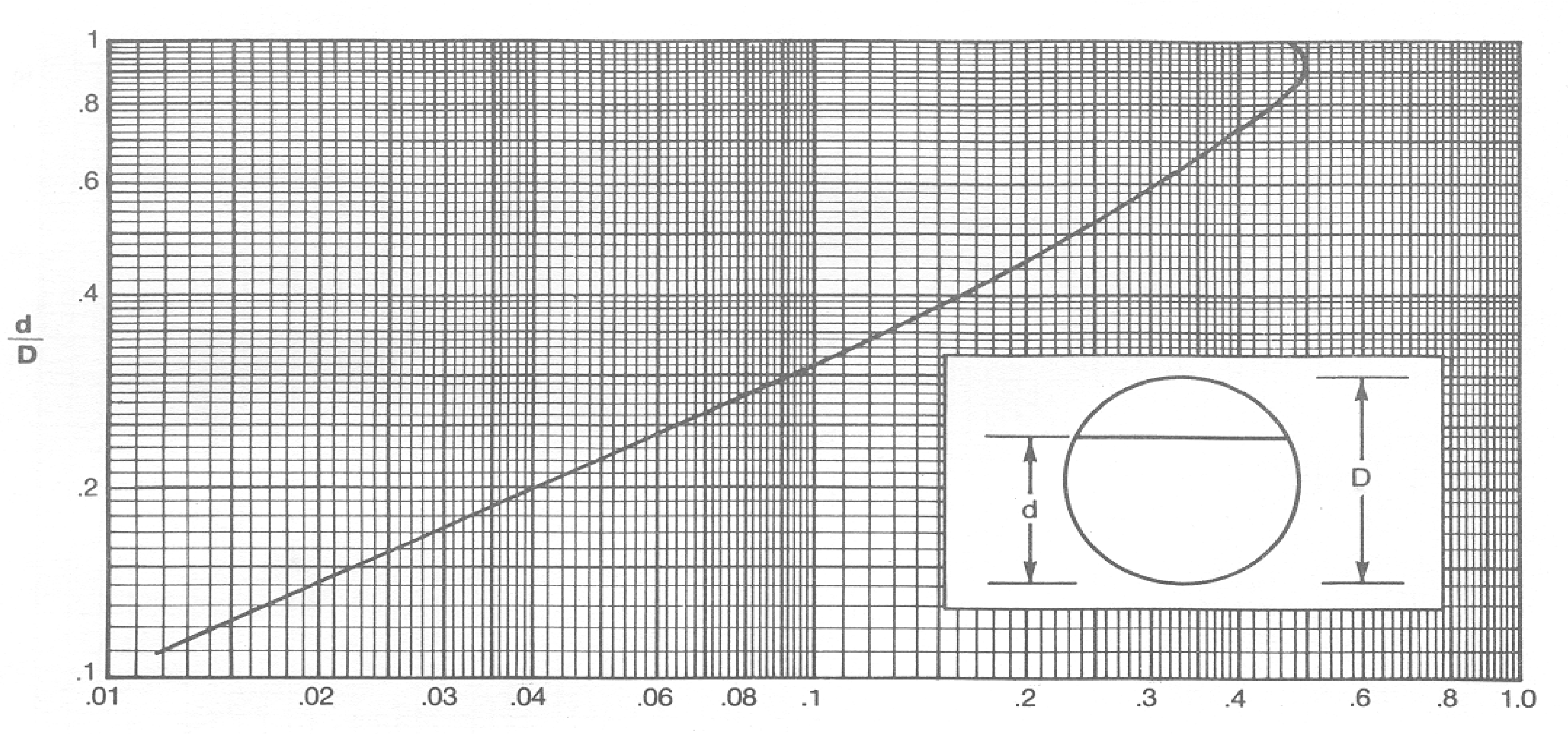
$$\text{Head loss [in feet]} = K\frac{V_o^{\;2}}{2g}$$

Where: K = loss factor (or coefficient)
V_o = flow velocity in the outlet pipe of the junction box. (ft/s)
g = gravitational constant = 32.2 ft/s^2

FHWA has printed the latest information on computing minor losses in HEC-22. FHWA continues to do research on minor losses. A report titled "Junction Loss Experiments: Laboratory Report" summarizes work that has been done more recently than the information published in HEC-22. The report and HEC-22 are available from the Internet at:

http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm

The Drainage Manual does not require that minor losses be calculated if the hydraulic gradient is kept 1.0' below the theoretical gutter elevation (1.13’ below the edge of pavement for sections with Type E or F curb and gutter). Nevertheless, it is important to calculate minor losses in high velocity situations and in long systems. As the velocity approaches 8 feet per second, the velocity head ($V^2/2g$) approaches one foot (64/64.4). The standard one foot of HGL clearance would be used up where the total loss coefficient, k = 1.0. For long systems the 1.0 foot of clearance could be used up by numerous small individual junction losses. For example, 10 junctions with 0.1' minor loss each.



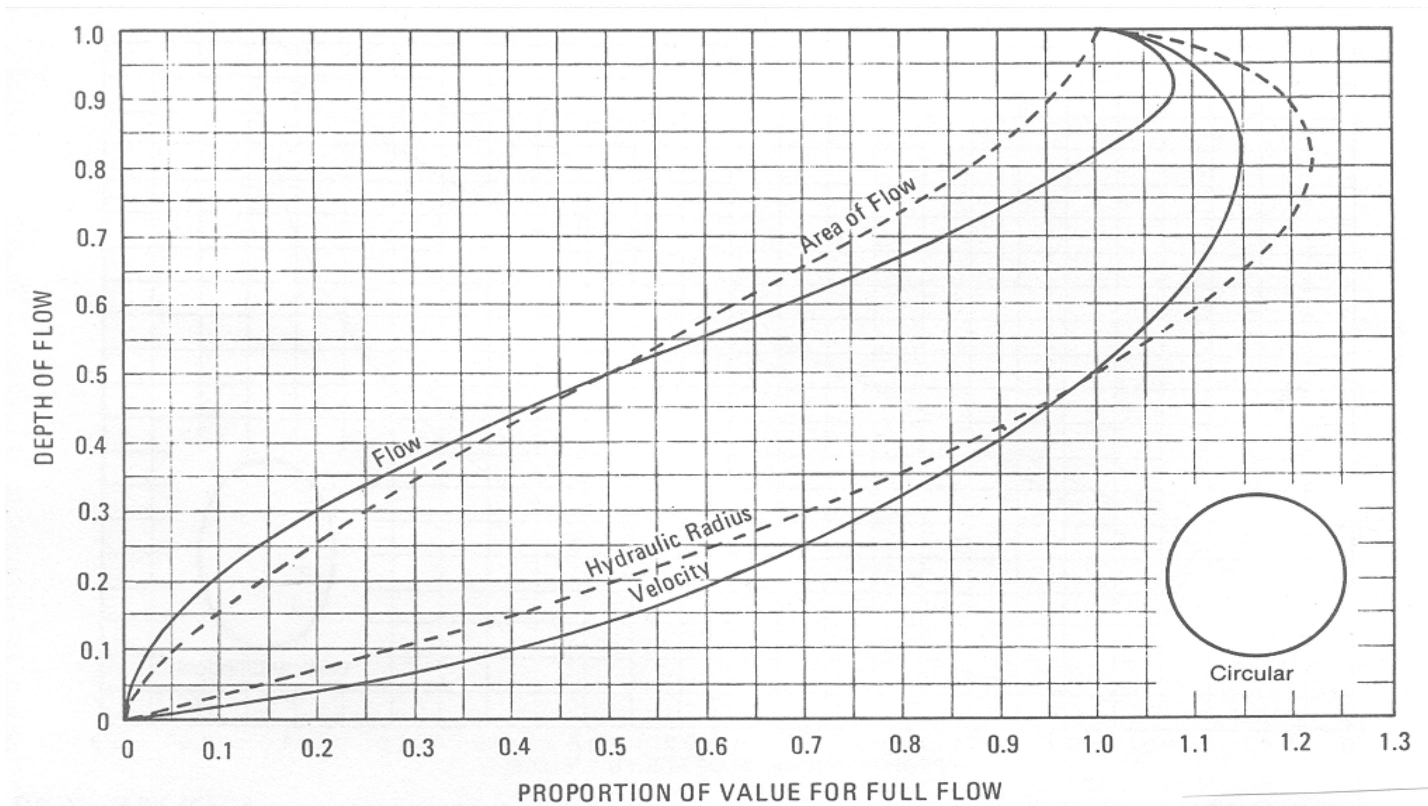
$$\frac{Qn}{D^{8/3} S^{1/2}}$$

Q = Flow Rate (cfs)

D = Pipe Diameter (ft)
S = Pipe Slope (ft/ft)
d = Normal Depth (ft)

Ref 1987 FDOT Drainage Manual

Figure 5-5 **Circular Pipe Partial Flow Capacity Chart**



Reference: American Concrete Pipe Association (1980).

Figure 5-6 Circular Pipe Relative Flow, Area, Hydraulic Radius, & Velocity for any Depth

Chapter 6 - Procedure

The following is a basic procedure for designing a storm drain system. You can vary slightly from the procedure and still develop an adequate design. With experience, you will develop short cuts and personal preference. The goal is to minimize pipe sizes while meeting the appropriate standards.

The numbers in parentheses **(xx)** refer to a space on the Storm Drain Tabulation Form.

IDENTIFY INLET LOCATIONS	
1. Define Overall Basin Draining to the Project	Using the drainage map, identify the overall watershed that drains to the project.
2. Determine the Outfalls and Divide the Overall Basin into Sub Basins.	This is typically done as a part of the stormwater management design.
3. For each Sub Basin, Select Inlet Locations.	
4. Determine the Drainage Area to Each Inlet	
5. Calculate Spread and Revise Inlet Location as Necessary.	
LAYOUT PIPES	
6. Connect pipes between the inlets to create a schematic of the piping system layout.	You will use the schematic of the piping system for the rest of the design procedure.

DETERMINE THE TOTAL "C•A" PRODUCT FOR EACH PIPE SECTION Begin filling out Storm Sewer Tabulation Form. Record the Inlet Types **(7)**, Inlet Locations **(3) (4) (5)**, Inlet Elevations **(19)**, Structure Numbers **(6)**, Incremental Areas **(9)**, C-Factors **(1)**, and Length **(8)** on the tabulation form. The incremental areas and C-factors are those used to calculate the spread.	
7. Add the areas that contribute flow to the downstream pipe.	This involves checking for all the upstream areas. Refer to the piping system schematic to ensure that all the areas are included. Record these in the space **(10)** on the tabulation form.
8. Multiply the subtotal areas by their respective C-factors.	Record the result in space **(11)** on the tabulation form.
9. Add the sub total (CA) values.	Record the total in space **(15)** on the tabulation form.
10. Repeat steps 7 through 9 for the entire system.	
PRELIMINARY HYDRAULIC GRADE LINE (HGL) SLOPE	
11. Estimate a Preliminary Hydraulic Grade Line Slope. This slope will be used as a guide for selecting the trial pipe size only. It will not control the final design.	For flat terrain, assume which inlet will be critical. The critical inlet will usually be the lowest inlet in the portion of the system farthest from the outlet. It may be simply the inlet farthest from the outlet. Use the following formula to calculate the slope. $\text{Slope} = \dfrac{\text{Critical Inlet Elev - DTW - 1 foot}}{\text{System Length between Outlet \& Critical Inlet}}$ For moderately sloped terrain, an average slope of the ground line along the project is usually acceptable for a preliminary HGL slope. For some systems there may be two or more distinct sections of the system that have noticeably different slopes. For these, calculating a preliminary HGL slope for each section is advised.

CALCULATE RUNOFF FLOW RATES The following is the beginning of an iterative process of calculating flow rates as you move down the system and calculating hydraulic grade line elevations as you move up the system. Steps 12 through 18 are done on each pipe segment beginning at the upper end of the system working down toward the outlet.	
12. Determine the t_c.	Record the value in space **(12)** on the tabulation form.
13. Determine the intensity.	Determine the intensity from the appropriate IDF curve using a storm duration equal to the t_c previously computed. Record the value in space **(14)** on the tabulation form.
14. Calculate total runoff for the pipe segment.	Multiply the total CA times the intensity. Record the value in space **(17)** on the tabulation form.
15. Select a pipe size.	For the first pass through the system select a diameter that has a full flow friction slope close to the preliminary HGL slope. The minimum pipe diameter will probably control the pipe size of the first few pipe sections. You will probably not find a pipe diameter that matches the preliminary HGL exactly. The objective is to maintain the standard HGL clearance at each inlet. Matching the preliminary HGL is merely a technique to begin selecting pipe diameters. Some pipe diameters will likely be revised later. Record the pipe size **(30) (31)** and associated Minimum Physical slope **(34)** on the tabulation form. Record the full flow friction slope as the hydraulic grade line slope **(32)** during the first pass down the system. The full flow friction slope will be used in the calculation of the hydraulic gradient.
16. Determine the pipe flow lines, fall, and physical slope.	The flow lines will usually be controlled by such things as cover requirements, structure clearances, and minimum physical pipe slope. Record the Flow Line Elevations **(25) (26)**, Crown Elevations **(23) (24)**, Physical Slope **(33)**, and Pipe Fall **(28)** on the tabulation form.
17. Calculate the flow velocity.	Actual Velocity: For the first pass through the system, assume full flow unless the pipe is obviously flowing part full. For subsequent passes through the system, use full flow velocity or velocity associated with normal depth, as appropriate. See discussion in Chapter 5. Record the value in space **(35)** on the tabulation form. Physical Velocity: Record the value in space **(36)** of the tabulation form.
18. Calculate time of flow in pipe section.	Divide the pipe length by the actual flow velocity. Record the value in space **(13)** on the tabulation form.
19. Repeat steps 12 through 18 for the entire system.	Check for peak flow from reduced area. See discussion in Chapter 2.

CALCULATE HYDRAULIC GRADE LINE (HGL) ELEVATION Steps 20 & 21 are done on each pipe segment beginning at the outlet and working up the system toward the most remote inlet.	
20. Determine the Lower End Hydraulic Gradient Elevation.	The Lower End HG for the outlet is the Design Tailwater (DTW). See discussion in Chapter 5. Record the value in space **(22)** on the tabulation form of the outlet pipe and in space **(41)**.
21. Determine Upper End HG Elevation, HGL Slope and HGL Fall.	See discussion in Chapter 5. Record the Upper End HG Elevation **(21)**. Record the HGL Clearance **(20)**. Where a pipe is flowing full, the full flow friction slope recorded in Step 15 is the Hydraulic Grade Line Slope **(32)**. The HG Fall **(27)** is calculated by multiplying the HGL Slope by the pipe length. Where a pipe is flowing part full and the Upper End HG is based on Normal depth as in Figure 5-2 C & D, the HG Slope and Fall recorded in Step 15 is not correct. Here, the HG Slope and Fall are not critical to the design process, but their values can be recorded as HG Fall **(27)** = Upper End HG - Lower End HG HG Slope **(32)** = HG Fall / pipe length.
Repeat steps 20 & 21 for the entire system. For the first pass through the system you may want to calculate the HGL elevation only along the main line from the outlet to the critical inlet. The flow rates and friction losses in the stub lines are usually small. Calculating the HG through the entire system (i.e., all the stubs) for the first iteration is acceptable, but may result in extra effort. For subsequent passes, calculate the HGL elevation for the entire system.	

COMPARE HYDRAULIC GRADE LINE (HGL) ELEVATION TO STANDARD	
22. Compare the HGL Elevation to the Standard and Adjust Pipe Diameters.	The current standard requires that the Hydraulic Gradient be at least 1 foot below the inlet elevations. For systems where the distance between the Hydraulic Gradient and the gutter elevation is greater than the standard, the diameter of one or more pipe segments may be reduced to raise the Hydraulic Gradient. Here, try to reduce the larger diameter pipe segments first as this will provide a greater cost reduction than reducing the size of the smaller diameter segments. For systems where the distance between the Hydraulic Gradient and the gutter elevation is less than the standard, you will need to increase the diameter of one or more pipe segments to lower the Hydraulic Gradient. Here, increase the smaller diameter pipe segments first as this will provide less of a cost increase than increasing the size of the larger diameter pipe segments. Look for “flow-pipe size” combinations that have substantial friction losses. For example, there is very little reduction in the losses by increasing the diameter of a 24” pipe that is carrying only 3 cfs. Alternatively, if another 24" pipe were carrying 15 cfs, increasing the pipe diameter could achieve a significant reduction in friction losses.
RECALCULATE THE RUNOFF AND HYDRAULIC GRADE LINE ELEVATION	
23. Return to Step 14 working the changes through the system.	When you have made enough iterations through the system such that any changes in diameters of pipe segments would cause the distance between the Hydraulic Gradient and the gutter elevation to be less than the standard, your design is essentially complete.

Note:

Examples 6.1 and 6.2 were created before the Plan Preparation Manual (Volume 2, Chapter 1.3) required that flow lines be shown to two decimal places and before the Drainage Manual required that HGL Clearance be provided in the storm tab. The examples have not been revised to reflect these changes. Although the examples have not been revised, they still represent a valid design procedure.

Example 6.1 Flat System - Determining Appropriate Pipe Sizes

Given:

- Inlets, Pipes, Runoff Coefficients & Details shown in Figure 6-1 and Table 6-1
- System discharges to a pond which stages to elevation 8.3 during a 3-year design storm.

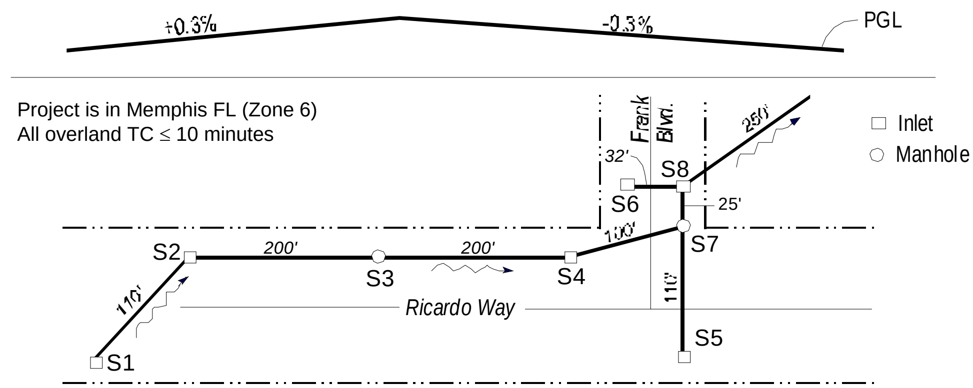


Figure 6-1 Example 6.1 – Given Information

LOCATION OF UPPER END			STRUCTURE NO.	TYPE OF STRUCTURE	LENGTH (ft)	DRAINAGE AREA (ac)		INLET ELEV (ft)	NOTES AND REMARKS
ALIGNMENT NAME									ZONE: 6
						C = 0.95			FREQUENCY (yrs): 3
STATION	DISTANCE (ft)	SIDE				C =			MANNING'S "n": 0.012
						C = 0.20			TAILWATER EL. (ft) 8.3
			UPPER			INCREMENT	TOTAL		All overland t_c < 10 min.
			LOWER						
Ricardo Way			1	P5	110	0.3		10.90	
40 + 80	46.5	R	2			0.05			
Ricardo Way			2	P5	200	0.2		11.10	
41 + 25	46.5	L	3			0.03			
Ricardo Way			3	MH	200	---		11.40	
43 + 25	44	L	4			---			
Ricardo Way			4	P5	100	0.4		11.10	
45 + 25	46.5	L	7			0.1			
Ricardo Way			5	P5	110	0.4		10.90	
46 + 00	46.5	R	7			0.5			
Frank Blvd			7	MH	25	---		10.50	
30 + 50	10	R	8			---			
Frank Blvd			6	P5	32	0.15		9.60	
30 + 75	16	L	8			0.5			
Frank Blvd			8	P5	250	0.25		9.60	
30 + 75	16	R	outlet			0.5			

Table 6-1

Find:

- The pipe sizes to meet the standard hydraulic gradient clearance of 1.13' to the inlet (edge of pavement) elevation.

1. Add the areas that contribute flow to each pipe segment. For each of the pipe segments the total impervious area (C=0.95) is obtained as follows.

Total Area $P_{1\text{-}2}$ = Inc. $Area_{S\text{-}1}$; no upstream pipes
= 0.3 ac.
Total Area $P_{2\text{-}3}$ = Inc. $Area_{S\text{-}2}$ + Total Area $P_{1\text{-}2}$
= 0.2 + 0.3 = 0.5 ac
Total Area $P_{3\text{-}4}$ = Inc. $Area_{S\text{-}3}$ + Total Area $P_{2\text{-}3}$
= 0.0 + 0.5 = 0.5 ac
Total Area $P_{4\text{-}7}$ = Inc. $Area_{S\text{-}4}$ + Total Area $P_{3\text{-}4}$
= 0.4 + 0.5 = 0.9 ac
Total Area $P_{5\text{-}7}$ = Inc. $Area_{S\text{-}5}$; no upstream pipes
= 0.4 ac.
Total Area $P_{7\text{-}8}$ = Inc. $Area_{S\text{-}7}$ + Tot. Area $P_{4\text{-}7}$ + Tot. Area $P_{5\text{-}7}$
= 0.0 + 0.9 + 0.4 = 1.3 ac
Total Area $P_{6\text{-}8}$ = Inc. $Area_{S\text{-}6}$; no upstream pipes
= 0.15 ac.
Total Area $P_{8\text{-}out}$= Inc. $Area_{S\text{-}8}$ + Tot. Area $P_{7\text{-}8}$ + Tot. Area $P_{6\text{-}8}$
= 0.25 + 1.3 + 0.15 = 1.7 ac

The same approach is applied to drainage areas associated with the pervious runoff coefficient. Table 6-2 is a partial tabulation form with the above information.

STRUCTURE NO.	DRAINAGE AREA (ac.)	
	C= 0.95	
	C=	
	C= 0.20	
UPPER	INCREMENT	TOTAL
LOWER		
1	0.3	0.3
2	0.05	0.05
2	0.2	0.5
3	0.03	0.08
3	---	0.5
4	---	0.08
4	0.4	0.9
7	0.1	0.18
5	0.4	0.4
7	0.5	0.5
7	---	1.3
8	---	0.68
6	0.15	0.15
8	0.5	0.5
8	0.25	1.7
outlet	0.5	1.68

Table 6-2

2. For each pipe section, multiply the total area associated with each runoff coefficient by the corresponding runoff coefficient to obtain the subtotal CA values.

$P_{1\text{-}2}$: 0.3 x 0.95 = 0.29
0.05 x 0.20 = 0.01
$P_{2\text{-}3}$: 0.5 x 0.95 = 0.48
0.08 x 0.20 = 0.02
Etc.

3. For each pipe section, add the subtotal CA values to obtain the Total CA.

$P_{1\text{-}2}$: 0.29 +0.01 = 0.3
$P_{2\text{-}3}$: 0.48 + 0.02 = 0.5
Etc.

Table 6-3 is a partial tabulation form with the above information.

ESTIMATE A PRELIMINARY HYDRAULIC GRADE LINE (HGL) SLOPE

4. Determine the design TW.
   The crown of the outlet pipe is not known at this time so we will use the given information about the stage (8.3 ft) of the stormwater management facility.

5. Assume which inlet will be critical.
   For this example we will assume that S-1 is critical. Elevation S-1 = 10.9 ft

6. Calculate a preliminary HGL slope
   For this example we will base the preliminary HGL slope on the following formula.

$$\text{Slope} = \frac{\text{Critical Inlet Elev} - \text{DTW} - 1\text{ foot}}{\text{System Length between Outlet \& Critical Inlet}}$$

The total pipe length is best seen in Figure 6-1.
110 + 200 + 200 + 100 + 25 + 250 = 885 ft
Prelim HGL Slope = 10.9 - 8.3 -1 / 885
= 0.0018 ft/ft
= 0.18 %

STRUCTURE NO.	DRAINAGE AREA (ac.)		SUB-TOTAL C•A	TOTAL C•A
UPPER	C= 0.95			
	C=			
	C= 0.20			
LOWER	INCREMENT	TOTAL		
1	0.3	0.3	0.29	0.30
2	0.05	0.05	0.01	
2	0.2	0.5	0.48	0.50
3	0.03	0.08	0.02	
3	---	0.5	0.48	0.50
4	---	0.08	0.02	
4	0.4	0.9	0.86	0.9
7	0.1	0.18	0.04	
5	0.4	0.4	0.38	0.48
7	0.5	0.5	0.1	
7	---	1.3	1.24	1.38
8	---	0.68	0.14	
6	0.15	0.15	0.14	0.24
8	0.5	0.5	0.1	
8	0.25	1.7	1.62	1.96
outlet	0.5	1.68	0.34	

Table 6-3

CALCULATE RUNOFF FLOW RATES (1st PASS DOWN THE SYSTEM)

Starting with pipe section P_{1-2},

7. Determine the time of concentration. [P_{1-2}]
   Since this is the first inlet in the system and it has an overland t_c of 10 minutes or less, use 10-minute minimum.

8. Determine the intensity. [P_{1-2}]
   From the IDF curve for zone 6 the 10-minute intensity is 6.5 in/hr.

9. Calculate the total runoff for the pipe section. [P_{1-2}]
   Q = Total CA (Step 3) times the intensity (previous step)
   Q = 0.3 x 6.5 = 1.95 cfs

10. Determine pipe size. [P_{1-2}] For the first pass, we assume full flow.
Using the hydraulic calculator, an 18" pipe is acceptable because the friction slope (0.03%) is flatter than the preliminary HGL slope (0.18%). The minimum physical slope is 0.15% (See discussion on page 40). Record the pipe size, and the minimum physical slope. Also, record the full flow friction slope as the HGL slope. Although it is not necessary to record the HGL slope at this step, it will be used later when moving up the system and calculating the hydraulic gradient. It may save time to record this while the hydraulic calculator is set for the flow and pipe size.

11. Determine the pipe flow lines, physical slope & fall. [P_{1-2}]
For this example, we will use 4.5' clearance between the inlet (edge of pavement) elevation and the flow line of an 18" pipe. (The minimum clearance for an 18" pipe in a precast Type P-5 structure is 4.2'. See Table 4-5.) Then:
Upper End Flow Line = 10.9 - 4.5 = 6.4 ft

For this example we will assume there are no constraints such as utilities that would prevent the pipe from being set at the minimum physical pipe slope (0.15%). Then:

Minimum pipe fall	= 110 ft x 0.15% = 0.17 ft
Pipe fall	= 0.2 ft (Minimum fall rounded up to nearest 0.1')
Physical Slope	= 0.2 ft / 110 ft = 0.18%
Lower End Flow Line	= 6.4 - 0.2 = 6.2 ft

> If this were an actual project, you should also check that the minimum cover heights of Drainage Manual Appendix E are satisfied. To simplify this example, we will assume that the adequate cover is provided.

12. Calculate the actual flow velocity. [P_{1-2}]
$Vel = Q / A \quad = 1.95 \text{ cfs} / \pi D^2/4)$
Full Flow cross sectional area is used for the first pass down the system. This is reasonable for a flat system like this example. If you know the pipe is flowing partially full use the average cross sectional flow area. See discussion on page 56 and the next example.

Using the hydraulic calculator, the velocity of 1.95 cfs flowing full through an 18" pipe is 1.1 fps.

Calculate the physical velocity. [P_{1-2}]
Using the hydraulic calculator, the full flow velocity for an 18” pipe sloped at 0.18% = 2.7 fps

13. Calculate the time of flow in pipe section. [P_{1-2}]
Time = Length / Actual Velocity
= 110 ft / 1.1 fps = 100 seconds = 1.7 minutes

A partially completed tabulation form is shown in Table 6-4.

STRUCTURE NUMBER	TYPE OF STRUCTURE	LENGTH (ft)	TIME OF CONCEN-TRATION (min)	TIME OF FLOW IN SECTION (min)	INTENSITY (iph)	TOTAL C • A	TOTAL FLOW (cfs)	INLET ELEV. (ft)	HYD. GRADIENT / CROWN / FLOW LINE: UPPER END ELEV. (ft)	HYD. GRADIENT / CROWN / FLOW LINE: LOWER END ELEV. (ft)	FALL (ft)	PIPE SIZE (IN.) RISE / SPAN	SLOPE (%) HYD. GRAD. / PHYSICAL / MIN. PHYS.	ACTUAL VELOCITY (FPS) / PHYSICAL VELOCITY (FPS)
UPPER / LOWER														
1	P5	110	10	1.7	6.5	0.30	1.95	10.90			0.2	18	.03	1.1
									7.9	7.7			.18	
2									6.4	6.2		18	.15	2.7

Table 6-4

For pipe section $P_{2\text{-}3}$,

14. Determine the Time of Concentration. [$P_{2\text{-}3}$]
    t_c overland ≤ 10 min.
    t_c system = 10 + 1.7 = 11.7 min. therefore
    t_c = 11.7 min.

15. Determine the intensity. [$P_{2\text{-}3}$]
    From the IDF curve for Zone 6 the 11.7-minute intensity is 6.1 in/hr.

16. Calculate the total runoff for the pipe section. [$P_{2\text{-}3}$]
    Flow rate = Total CA x Intensity
    = 0.5 x 6.1 = 3.1

17. Determine pipe size. [$P_{2\text{-}3}$]
    Using the hydraulic calculator, an 18" pipe is acceptable because the friction slope (0.07%) is less than the preliminary HGL slope (0.18%). As done for the previous pipe section, record the pipe size, and the minimum physical slope. Also, record the friction slope as the HGL slope.

18. Determine the pipe flow lines, physical slope & fall. [$P_{2\text{-}3}$]
    Since this inlet is higher than S-1, the potential conflict with the inlet top will not control the flow lines. For this example, we will attempt to match flow line elevations across structures. Therefore:
    Upper End FL = 6.2 (Previous pipe section downstream flow line)
    Minimum Pipe fall = length x min phys. slope
    = 200 ft x 0.15% = 0.3 ft
    Pipe fall = 0.3 ft
    Physical Slope = 0.3 ft / 200 ft = 0.15%
    Lower End Flow Line = 6.2 - 0.3 = 5.9 ft

19. Calculate the actual flow velocity. [$P_{2\text{-}3}$]
    Vel = Q / A = 1.95 cfs / (π D^2/4).

Full Flow cross sectional area is used for the first pass through the system. Using the hydraulic calculator, the velocity of 3.1 cfs flowing full through an 18" pipe is approximately 1.8 fps.

Calculate the physical velocity. [$P_{2\text{-}3}$]
Using the hydraulic calculator, the full flow velocity for an 18” pipe sloped at 0.15% = 2.5 fps

20. Calculate the time of flow in pipe section. [$P_{2\text{-}3}$]
    Time = Length / Actual Velocity
    = 200 ft / 1.8 fps = 111 seconds = 1.9 minutes

A partially completed tabulation form is shown in Table 6-5.

STRUCTURE NO. (UPPER / LOWER)	TYPE OF STRUCTURE	LENGTH (ft)	TIME OF CONCENTRATION (min)	TIME OF FLOW IN SECTION (min)	INTENSITY (iph)	TOTAL C • A	TOTAL FLOW (cfs)	INLET ELEV. (ft)	HYD. GRADIENT / CROWN / FLOW LINE: UPPER END ELEV. (ft)	LOWER END ELEV. (ft)	FALL (ft)	PIPE SIZE (IN): RISE / SPAN	SLOPE (%): HYD. GRAD. / PHYSICAL / MIN. PHYS.	ACTUAL VELOCITY (FPS) / PHYSICAL VELOCITY (FPS)
1	P5	110	10	1.7	6.5	0.30	1.95	10.90			0.2	18	0.03	1.1
									7.9	7.7			0.18	
2									6.4	6.2		18	0.15	2.7
2	P5	200	11.7	1.9	6.1	0.50	3.1	11.10			0.3	18	0.07	1.8
									7.7	7.4			0.15	
3									6.2	5.9		18	0.15	2.5

Table 6-5

Steps 14 through 20 are repeated for the remaining pipe sections. Situations that are different from the above pipe sections are discussed below. Table 6-6 shows the results of doing these steps for the entire system.

Pipe Section $P_{3\text{-}4}$

The manhole contributes no additional flow to the system, nor does it combine flow from several pipes. The time of concentration and the intensity are not applicable. The flow through the pipe section is the same as the upstream pipe section.

Pipe Section $P_{4\text{-}7,}$

The time of concentration is 11.7 + 1.9 +1.9 = 15.5 minutes.

Pipe Sections $P_{5\text{-}7}$ and $P_{6\text{-}8}$

They receive only overland flow like pipe section $P_{1\text{-}2}$, thus their time of concentration is based on overland flow time. Their flow lines are determined by matching the flow lines of the downstream structure, using the minimum physical slope to the upstream structure, and rounding to the nearest 0.1' such that the minimum slope is maintained.

Pipe Section P_{7-8}

This is similar to similar to pipe section P_{3-4} in that the manhole contributes no flow. It is different from pipe section P_{3-4} in that two pipes drain to the manhole. Because of this difference, the pipe section is treated like the other inlets along the main line. The t_c, intensity, and flow are calculated for the section. The time of concentration is 15.5 + 0.6 = 16.1 minutes.

As stated in Step 18, we will attempt to match flow lines across structures for this example. The Upper End FL is set to match the Lower End flow line of pipe section P_{4-7}. The Lower End FL is set to match the FL of S-8.

Pipe Section P_{8-out}

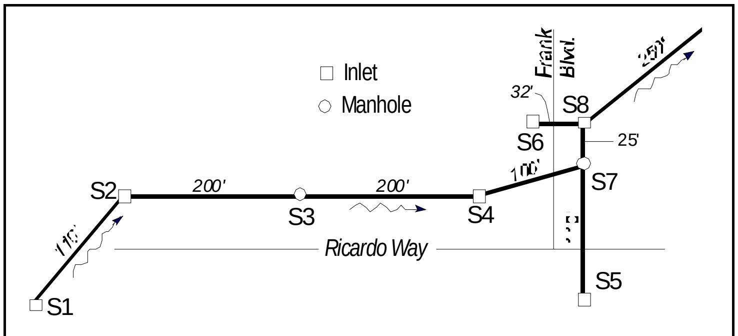
For this example, we will use 5.1' clearance between the inlet (edge of pavement) elevation and the flow line of a 24" pipe. (The minimum clearance for a 24" pipe in a precast Type P-5 structure is 4.7'. See Table 4-5.) Then Upper End FL = 9.6 - 5.1 = 4.5'. The Lower End FL is set to match the minimum physical slope with the FL rounded to the closest 0.1' such that the minimum slope is maintained.

STRUCTURE NO. (UPPER / LOWER)	TYPE OF STRUCTURE	LENGTH (ft)	TIME OF CONCENTRATION (min)	TIME OF FLOW IN SECTION (min)	INTENSITY (iph)	TOTAL C • A	TOTAL FLOW (cfs)	INLETELEV. (ft)	HYD. GRADIENT / CROWN / FLOW LINE: UPPER END ELEV. (ft)	LOWER END ELEV. (ft)	FALL (ft)	PIPE SIZE (IN): RISE / SPAN	SLOPE (%): HYD. GRAD. / PHYSICAL / MIN. PHYS.	ACTUAL VELOCITY (FPS) / PHYSICAL VELOCITY (FPS)
1	P5	110	10	1.7	6.5	0.30	1.95	10.90				18	.03	**1.1**
									7.9	7.7	0.2		.18	
2									6.4	6.2		18	.15	**2.7**
2	P5	200	11.7	1.9	6.1	0.50	3.1	11.10				18	0.07	**1.8**
									7.7	7.4	0.3		0.15	
3									6.2	5.9		18	0.15	**2.5**
3	MH	200	N/A	1.9	N/A	0.50	3.1	11.40				18	0.07	**1.8**
									7.4	7.1	0.3		0.15	
4									5.9	5.6		18	0.15	**2.5**
4	P5	100	15.5	0.6	5.4	0.9	4.9	11.10				18	0.18	**2.8**
									7.1	6.9	0.2		0.2	
7									5.6	5.4		18	0.15	**2.9**
5	P5	110	10	1.7	6.5	0.48	3.1	10.90				18	0.07	**1.8**
									7.1	6.9	0.2		0.18	
7									5.6	5.4		18	0.15	**2.7**
7	MH	25	16.1	0.2	5.3	1.38	7.3	10.50				24	0.09	**2.4**
									7.4	6.5	0.9		3.6	
8									5.4	4.5		24	0.1	**14.8**
6	P5	32	10	0.4	6.5	0.24	1.56	9.60				18	0.02	**1.3**
									6.1	6.0	0.1		0.3	
8									4.6	4.5		18	0.15	**3.5**
8	P5	250	16.3	1.9	5.3	1.96	10.4	9.60				24	0.18	**3.4**
									6.5	6.2	0.3		0.12	
outlet									4.5	4.2		24	0.1	**2.7**

Table 6-6
Results of 1st pass down the System

21. Check for peak flow from reduced area.
Reviewing the size and runoff coefficient for each drainage area, it does not appear that most of the area, nor most of the imperviousness is concentrated near the lower end of the system. Thus, one would not expect to have peak flow from reduced area and detailed calculations would not be necessary. For this example, we will check it just to demonstrate an approach.

From the schematic, it appears that a logical reduced area would be area flowing overland to S4, S5, S6, & S8. The overland t_c to S4 was given as 10 minutes. So let's apply a 10-minute storm to pipe section $P_{4\text{-}7}$. Doing so reduces the contributing area from S3.



Schematic

An approach to finding the reduced contributing area is to multiply the area (or the CA product) from S3 by the ratio of the Times of Concentration. From Table 6-6, the t_c for the flow from S3 is 15.5 minutes. So reduce the Total CA from S3 by the ratio 10 / 15.5 or 0.65.

From Table 6-6 the Total CA from S3 = 0.5.

So the Total CA from S3 <u>is reduced by</u>: 0.5 (1 - 0.65) = 0.18 This value is not the reduced CA; it is the amount the Total CA is reduced by.

Reducing the Total CA for pipe section $P_{4\text{-}7}$ by this amount yields: 0.9 - 0.18 = 0.72

The 3-year intensity for a 10 minute storm = 6.5 in/hour.

The flow in the pipe downstream of S4 = CAi = 0.72 x 6.5 = 4.7 cfs.

This is less than the 4.9 cfs calculated for the entire contributing area for $P_{4\text{-}7}$ as shown in Table 6-6. So peak flow in pipe section P $_{4\text{-}7}$ does not result from reduced area. Although other pipe sections could be checked for peak flow from reduced area, this effort shown above is acceptable for this system.

CALCULATE THE HYDRAULIC GRADE LINE ELEVATION (1st pass up the system)

For pipe section $P_{8\text{-out}}$:

22. Determine the Lower End HG elevation. [$P_{8\text{-out}}$]
For the outlet pipe, the Lower End HG is the design tailwater. For this example the design tailwater is the higher of A) the crown of the pipe (El. 6.2 ft) or B) the peak stage of the stormwater facility during the storm drain design event (El. 8.3 ft). Thus, the Lower End HG = 8.3 ft

23. Determine the Upper End HG elevation. [$P_{8\text{-out}}$]
For this example the Lower End HG submerges the entire pipe section; therefore,
Upper End HG El. = Lower End HG El. + Full Flow Friction Loss.
& Full Flow Friction Loss = Full Flow Friction Slope x Pipe length
= 0.18% x 250 ft = 0.45 ft

The full flow friction slope was previously recorded as the hydraulic gradient slope in Table 6-6 when we moved down the system calculating flow rates.

Then Upper End HG El. = Lower End HG El. + Full Flow Friction Loss
= 8.3 + 0.45 = 8.75 ft See table below.

Pipe sections $P_{6\text{-}8}$ and $P_{5\text{-}7}$ are stubs and their hydraulic gradient will not be calculated during the 1st pass up the system.

STRUCTURE NO.	LENGTH (ft)	HYD. GRADIENT / CROWN / FLOW LINE			SLOPE (%)
UPPER / LOWER		UPPER ELEV (ft)	LOWER ELEV (ft)	FALL (ft)	HYD. GRAD. / PHYSICAL / MIN. PHYS.
8	250	8.75	8.3	0.45	0.18
		6.5	6.2	0.3	0.12
outlet		4.5	4.2		0.1

Table 6-7

For pipe section $P_{7\text{-}8}$

24. Determine the Lower End HG elevation.
The downstream pipe Upper End HG Elevation (8.75 ft) is higher than the Lower End Crown Elevation (6.5 ft); therefore, Lower End HG Elevation = downstream pipe upper end HG = 8.75 ft

25. Determine the Upper End HG elevation. [$P_{7\text{-}8}$]

For this example the Lower End HG submerges the entire pipe section; therefore,
Upper End HG El. = Lower End HG El. + Full Flow Friction Loss.
& Full Flow Friction Loss = Full Flow Friction Slope x Pipe length
= 0.09% x 25 ft = 0.02 ft
The full flow friction slope was previously recorded as the hydraulic gradient slope in Table 6-6.
Then Upper End HG El. = Lower End HG El. + Full Flow Friction Loss
= 8.75 + 0.02 = 8.77 ft

The same steps are repeated for the remaining mainline pipe sections. Table 6-8 shows the results of doing these steps for the entire system.

STRUCTURE NO. (UPPER / LOWER)	LENGTH (ft)	TOTAL FLOW (cfs)	INLET ELEV. (ft)	HYD. GRADIENT / CROWN / FLOW LINE: UPPER END ELEV (ft)	LOWER END ELEV (ft)	FALL (ft)	PIPE SIZE (IN): RISE / SPAN	SLOPE (%): HYD. GRAD. / PHYSICAL / MIN. PHYS.
1	110	1.95	10.90	9.26	9.23	0.03	18	0.03
				7.9	7.7	0.2		0.18
2				6.4	6.2		18	0.15
2	200	3.1	11.10	9.23	9.09	0.14	18	0.07
				7.7	7.4	0.3		0.15
3				6.2	5.9		18	0.15
3	200	3.1	11.40	9.09	8.95	0.14	18	0.07
				7.4	7.1	0.3		0.15
4				5.9	5.6		18	0.15
4	100	4.9	11.10	8.95	8.77	0.18	18	0.18
				7.1	6.9	0.2		0.20
7				5.6	5.4		18	0.15
5	110	3.1	10.90				18	0.07
				7.1	6.9	0.2		0.18
7				5.6	5.4		18	0.15
7	25	7.3	10.50	8.77	8.75	0.02	24	0.09
				7.4	6.5	0.9		3.60
8				5.4	4.5		24	0.10
6	32	1.56	9.60				18	0.02
				6.1	6.0	0.1		0.30
8				4.6	4.5		18	0.15
8	250	10.4	9.60	8.75	8.3	.45	24	0.18
				6.5	6.2	0.3		0.12
outlet				4.5	4.2		24	0.10

Table 6-8 Results of 1st Pass up the System

26. COMPARE THE HYDRAULIC GRADIENT TO THE STANDARD AND ADJUST PIPE SIZES

The standard clearance of 1.13' between the hydraulic gradient and the inlet elevation (edge of pavement) is not met for S-8 and probably not met for S-6. The remaining inlets have adequate clearance. Increasing the size of the outlet pipe $P_{8\text{-}out}$ to 30" will reduce the hydraulic gradient at S-6 and S-8 so we will try that. To reduce costs, we will also try reducing the pipe size of section $P_{7\text{-}8}$ to 18".

27. CALCULATE THE HYDRAULIC GRADIENT USING THE CHANGED PIPE SIZES

Table 6-9 shows the new slopes and the recalculated hydraulic gradient for the entire system.

STRUCTURE NO. (UPPER / LOWER)	LENGTH (ft)	TOTAL FLOW (cfs)	INLET ELEV. (ft)	HYD. GRADIENT / CROWN / FLOW LINE: UPPER END ELEV (ft)	LOWER END ELEV (ft)	FALL (ft)	PIPE SIZE (IN) RISE / SPAN	SLOPE (%) HYD. GRAD. / PHYSICAL / MIN. PHYS.
1	110	1.95	10.90	9.04	9.01	0.03	18	0.03
				7.9	7.7	0.20		0.18
2				6.4	6.2		18	0.15
2	200	3.1	11.10	9.01	8.87	0.14	18	0.07
				7.7	7.4	0.30		0.15
3				6.2	5.9		18	0.15
3	200	3.1	11.40	8.87	8.73	0.14	18	0.07
				7.4	7.1	0.30		0.15
4				5.9	5.6		18	0.15
4	100	4.9	11.10	8.73	8.55	0.18	18	0.18
				7.1	6.9	0.20		0.20
7				5.6	5.4		18	0.15
5	110	3.1	10.90	8.63	8.55	0.08	18	0.07
				7.1	6.9	0.20		0.18
7				5.6	5.4		18	0.15
7	25	7.3	10.50	8.55	8.45	0.10	18	0.40
				6.9	6.0	0.90		3.60
8				5.4	4.5		18	0.15
6	32	1.56	9.60	8.46	8.45	0.01	18	0.02
				6.1	6.0	0.10		0.30
8				4.6	4.5		18	0.15
8	250	10.4	9.60	8.45	8.3	0.15	30	0.06
				7.0	6.8	0.20		0.08
outlet				4.5	4.3		30	0.08

Table 6-9 **Results of 2nd Pass up the System**

28. COMPARE THE HG TO THE STANDARD AND ADJUST PIPE SIZES
From Table 6-9, the standard 1.13' of clearance between the hydraulic gradient and the inlet elevation (edge of pavement) exists throughout the system.

29. RECALCULATE THE FLOW

Changing pipe sizes changes the velocity, thus changing the time of flow in section and time of concentration. These changes affect only the changed pipes and the pipes downstream of the changed pipes. For this example, only pipe sections $P_{7\text{-}8}$ and $P_{8\text{-out}}$ are affected.

The increased velocity in pipe section $P_{7\text{-}8}$ reduced the time of flow in the pipe by only 0.1 minute because the pipe is so short. As a result, the time of concentration of the outlet pipe was reduced by only 0.1 minute from 16.3 to 16.2 minutes. It is hard to read a change in the intensity from the IDF curve for a change in t_c of 0.1 minute. Although we changed the size of the two pipes, there was no noticeable change to the flow rate in the system.

A completed tabulation form is shown in Table 6-10.

Example 6-1

NOTES AND REMARKS
ZONE: 6
FREQUENCY (Yrs) **: 3**
MANNING'S "n" : 0.012
TAILWATER EL. (ft): 8.3
All overland t_c < 10 Min.

LOCATION OF UPPER END / ALIGNMENT NAME / STATION	DISTANCE (ft)	SIDE	STRUCTURE NO. (UPPER / LOWER)	TYPE OF STRUCTURE	LENGTH (ft)	DRAINAGE AREA (ac) INCRE-MENT (C = 0.95 / C = / C = 0.20)	DRAINAGE AREA (ac) TOTAL	C • A SUBTOTAL	TIME OF CONCENTRATION (MIN)	TIME OF FLOW IN SECTION (MIN)	INTENSITY (iph)	C • A TOTAL	TOTAL FLOW (cfs)	INLET ELEV. (ft)	HYD. GRADIENT CROWN / FLOW LINE: UPPER END ELEV (ft)	LOWER END ELEV (ft)	FALL (ft)	PIPE SIZE (IN) RISE / SPAN	SLOPE (%) HYD. GRAD / PHYSICAL / MIN. PHYS.	ACTUAL VELOCITY (FPS) / PHYSICAL VELOCITY (FPS)
Ricardo Way			1			**0.3**	**0.3**	**0.29**							**9.04**	**9.01**	**0.03**	**18**	**0.03**	**1.1**
				P5	**110**				**10**	**1.7**	**6.5**	**0.3**	**1.95**	**10.90**	**7.9**	**7.7**	**0.2**		**0.18**	
40 + 80	**46.5**	**R**	**2**			**0.05**	**0.05**	**0.01**							**6.4**	**6.2**			**0.15**	**2.7**
Ricardo Way			**2**			**0.2**	**0.5**	**0.48**							**9.01**	**8.87**	**0.14**	**18**	**0.07**	**1.8**
				P5	**200**				**11.7**	**1.9**	**6.1**	**0.5**	**3.1**	**11.10**	**7.7**	**7.4**	**0.3**		**0.15**	
41 + 25	**46.5**	L	**3**			**0.03**	**0.08**	**0.02**							**6.2**	**5.9**			**0.15**	**2.5**
Ricardo Way			**3**			**---**	**0.5**	**0.48**							**8.87**	**8.73**	**0.14**	**18**	**0.07**	**1.8**
				MH	**200**				**N/A**	**1.9**	**N/A**	**0.5**	**3.1**	**11.40**	**7.4**	**7.1**	**0.3**		**0.15**	
43 + 25	**44**	L	**4**			**---**	**0.08**	**0.02**							**5.9**	**5.6**			**0.15**	**2.5**
Ricardo Way			**4**			**0.4**	**0.9**	**0.86**							**8.73**	**8.55**	**0.18**	**18**	**0.18**	**2.8**
				P5	**100**				**15.5**	**0.6**	**5.4**	**0.9**	**4.9**	**11.10**	**7.1**	**6.9**	**0.2**		**0.2**	
45 + 25	**46.5**	L	**7**			**0.1**	**0.18**	**0.04**							**5.6**	**5.4**			**0.15**	**2.9**
Ricardo Way			**5**			**0.4**	**0.4**	**0.38**							**8.63**	**8.55**	**0.08**	**18**	**0.07**	**1.8**
				P5	**110**				**10**	**1.0**	**6.5**	**0.48**	**3.1**	**10.90**	**7.1**	**6.9**	**0.2**		**0.18**	
46 + 00	**46.5**	R	**7**			**0.5**	**0.5**	**0.1**							**5.6**	**5.4**			**0.15**	**2.7**
Frank Blvd.			**7**			**---**	**1.3**	**1.24**							**8.55**	**8.45**	**0.1**	**18**	**0.40**	**4.1**
				MH	**25**				**16.1**	**0.1**	**5.3**	**1.38**	**7.3**	**10.50**	**6.9**	**6.0**	**0.9**		**3.6**	
30 + 50	**10**	R	**8**			**---**	**0.68**	**0.14**							**5.4**	**4.5**			**0.15**	**12.3**
Frank Blvd.			**6**			**0.15**	**0.15**	**0.14**							**8.46**	**8.45**	**0.01**	**18**	**0.02**	**1.3**
				P5	**32**				**10**	**0.4**	**6.5**	**0.24**	**1.56**	**9.60**	**6.1**	**6.0**	**0.1**		**0.3**	
30 + 75	**16**	L	**8**			**0.5**	**0.5**	**0.1**							**4.6**	**4.5**			**0.15**	**3.5**
Frank Blvd			**8**			**0.25**	**1.7**	**1.62**							**8.45**	**8.3**	**0.15**	**30**	**0.06**	**2.2**
				P5	**250**				**16.2**	**1.9**	**5.3**	**1.96**	**10.4**	**9.60**	**7.0**	**6.8**	**0.2**		**0.08**	
30 + 75	**16**	R	**Outlet**			**0.5**	**1.68**	**0.34**							**4.5**	**4.3**			**0.08**	**2.5**

Table 6-10

Example 6.2 Steep System - Determining Appropriate Pipe Sizes

Given:

- Inlets, Pipes, Runoff Coefficients & Details in Figure 6-6 and Table 6-11.
- Designer chooses to match crown elevations across structures.

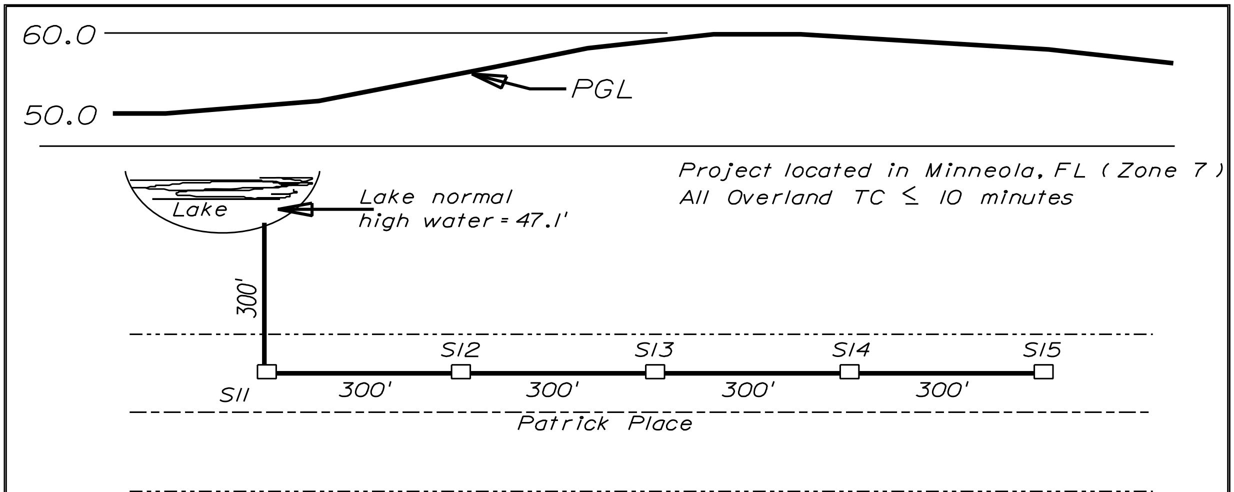


Figure 6-6 Example 6.2 - Given Information

LOCATION OF UPPER END			STRUCTURE NO.	TYPE OF STRUCTURE	LENGTH (ft)	DRAINAGE AREA (ac)		INLET ELEV. (ft)	NOTES AND REMARKS
ALIGNMENT NAME						C =			ZONE: 7
						C = **0.80**			
						C =			FREQUENCY (Yrs): 3
STATION	DISTANCE (ft)	SIDE				INCRE-MENT	TOTAL		MANNING'S "n": 0.012
									TAILWATER EL. (ft): 47.1
			UPPER						All overland $t_c \le$ 10 min.
			LOWER						
Patrick Place			**15**	**P1**	**300**	**0.2**		**59.70**	
			14						
Patrick Place			**14**	**P1**	**300**	**1.0**		**59.80**	
			13						
Patrick Place			**13**	**P1**	**300**	**0.6**		**59.00**	
			12						
Patrick Place			**12**	**P1**	**300**	**0.5**		**54.50**	
			11						
Patrick Place			**11**	**P1**	**300**	**1.1**		**50.50**	
			outlet						

Table 6-11

Example 6.2 – Given Information

Find:

- The pipe sizes to meet the standard hydraulic gradient clearance of 1.13' to the inlet (edge of pavement) elevation.

1. Add the areas that contribute flow to each pipe segment. For each of the pipe segments the total area is obtained as in Example 6.1.

2. For each pipe section, multiply the total area associated with each runoff coefficient by the corresponding runoff coefficient to obtain the subtotal CA values.

3. For each pipe section, add the subtotal CA values to obtain the total CA value.

Table 6-12 is a partial tabulation form complete with the information from the first three steps.

STRUCTURE NO. UPPER	STRUCTURE NO. LOWER	DRAINAGE AREA (ac)	INCREMENT	TOTAL	SUB-TOTAL C • A	TOTAL C • A
15	14	C =				0.16
		C = 0.80	0.2	0.2	0.16	
		C =				
14	13	C =				0.96
		C = 0.80	1	1.2	0.96	
		C =				
13	12	C =				1.44
		C = 0.80	0.6	1.8	1.44	
		C =				
12	11	C =				1.84
		C = 0.80	0.5	2.3	1.84	
		C =				
11	outlet	C =				2.72
		C = 0.80	1.1	3.4	2.72	
		C =				

Table 6-12

ESTIMATE A PRELIMINARY HGL SLOPE

4. Determine the design TW.
   The crown of the outlet pipe is not known at this time so we will use the given information about the lake stage. DTW = 47.1.

5. Assume which inlet will be critical. For this example we will assume that S -15 is critical.

6. For this example we will base the preliminary HGL slope on the following formula.

$$\text{Slope} = \frac{\text{Critical Inlet Elev - DTW - 1 foot}}{\text{System Length between Outlet \& Critical Inlet}}$$

Slope = (59.7 - 47.1 - 1) / 1500 = 0.8%

CALCULATE RUNOFF FLOW RATES
FIRST PASS DOWN THE SYSTEM

Starting with pipe section $P_{15\text{-}14}$,

7. Determine the time of concentration. [$P_{15\text{-}14}$]
   Since this is the first inlet in the system and it has an overland t_c of 10 minutes or less, use 10-minute minimum.

8. Determine the intensity. [$P_{15\text{-}14}$]
From the IDF curve for Zone 7 the 10-minute intensity is 6.5 in/hr.

9. Calculate the flow rate for the pipe section. [$P_{15\text{-}14}$]
Q = Total CA (Step 3) times the intensity (previous step)
Q = 0.16 x 6.5 = 1.04 cfs

10. Determine pipe size. [$P_{15\text{-}14}$] *For the first pass we are assuming full flow.*
Using the hydraulic calculator, an 18" is acceptable because the friction slope (<0.04) is flatter than the preliminary HGL slope. The minimum physical slope is 0.15% (See discussion on page 40.) Record the pipe size, and the minimum physical slope. Also, record the full flow friction slope as the HGL slope. For this flow rate through an 18" pipe the friction loss is so small that the Department's hydraulic calculator does not show the slope. The loss could be calculated from the equation on page 51, but for now we will record the HGL slope as zero.

11. Determine the pipe flow lines, physical slope & fall. [$P_{15\text{-}14}$]
For this example, we will use 4.5' clearance between the inlet (edge of pavement) elevation and the flow line of an 18" pipe. (The minimum clearance for standard precast structures is 4.2'. See Table 4-5.) Then:
Upper End Flow Line = 59.7 – 4.5 = 55.2 ft

For this example we will assume there are no constraints such as utilities that would prevent the pipe from being set at the minimum physical pipe slope (0.15%). Then:

Minimum pipe fall	= 300 ft x 0.15% = 0.45 ft
Pipe fall	= 0.5 ft (Minimum fall rounded up to nearest 0.1')
Physical Slope	= 0.5 ft / 300 ft = 0.167%
Lower End Flow Line	= 55.2 - 0.5 = 54.7 ft

> If this were an actual project, you should also check that the minimum cover heights in Appendix E of the Drainage Manual are satisfied. To simplify this example, we will assume that the adequate cover is provided.

12. Calculate the actual flow velocity. [$P_{15\text{-}14}$]
$V = Q / A = 1.04 \text{ cfs} / (\pi D^2/4)$
Full Flow cross sectional area used for the first pass through the system.
Using the hydraulic calculator, the velocity of 1.04 cfs flowing full through an 18" pipe is 0.59 fps.

Calculate the physical velocity. [$P_{15\text{-}14}$]

Using the hydraulic calculator, the full flow velocity for an 18" pipe sloped at 0.17% = 2.6 fps

13. Calculate the time of flow in pipe section. [$P_{15\text{-}14}$]
Time = Length / Actual Velocity

= 300 ft / 0.59 fps = 508 seconds = 8.5 minutes

A partially completed tabulation form with the information from this pipe is shown below.

STRUCTURE NO. (UPPER / LOWER)	LENGTH (ft)	TIME OF CONCEN-TRATION (MIN)	TIME OF FLOW IN SECTION (MIN)	INTENSITY (iph)	TOTAL C • A	TOTAL FLOW (cfs)	INLET ELEV. (ft)	HYD. GRADIENT / CROWN / FLOW LINE: UPPER END ELEV (ft)	HYD. GRADIENT / CROWN / FLOW LINE: LOWER END ELEV (ft)	FALL (ft)	PIPE SIZE (IN): RISE / SPAN	SLOPE (%): HYD. GRAD. / PHYSICAL / MIN. PHYS.	ACTUAL VELOCTIY (FPS) / PHYSICAL CELOCITY (FPS)
15	300	10	8.5	6.5	0.16	1.04	59.70				18	0	.59
								56.7	56.2	0.5		.167	
14								55.2	54.7		18	.15	2.6

Table 6-13

Steps 7 through 13 are repeated for the remaining pipe sections. We have assumed that all the pipes are flowing full for this pass down the system. Situations that are different from the above pipe section are discussed below.

For Pipe Section $P_{14\text{-}13}$:

The time of concentration is 10+8.5 = 18.5 minutes.
The upper end flow line = 54.7 (set by matching the crowns across the structure).
The lower end flow line = 54.7 - 0.5 = 54.2 (set by minimum pipe slope as was done for $P_{15\text{-}14}$).

For Pipe Section $P_{13\text{-}12}$:

The time of concentration is 18.5 + 1.8 = 20.3 minutes.
The upper flow line = 54.2 (set by matching the crowns across the structure).
The lower end flow line = S12 gutter elev. - inlet clearance = 54.5 - 4.5 = 50.0
The physical slope = (54.2 - 50.0) / 300 = 1.4%.

For Pipe Section $P_{12\text{-}11}$:

The time of concentration is 20.3 + 1.3 = 21.6 minutes.
The upper flow line = 50.0 (set by matching the crowns across the structure).
The lower end flow line = S11 gutter elev. - inlet clearance = 50.5 - 4.5 = 46.0
The physical slope = (50.0 - 46.0) / 300 = 1.33%.

For Pipe Section $P_{11\text{-}out}$:

Size could be 18" or 24" based on comparing the full flow friction loss slope to the preliminary HGL slope. Try 18" since the other pipes seem oversized.
The time of concentration is 21.6 + 1.0 = 22.6 minutes.
The upper flow line = 46.0 (set by matching the crowns across the structure).
Several factors may control the lower end flow line such as but not limited to cover requirements under roads around the lake, the lake bottom elevation, purposely submerging the outlet to minimize potential erosion at the outlet. For this example, we arbitrarily chose 44.5'.
The physical slope = (46.0 - 44.5) / 300 = 0.5%.

The full flow friction slope was recorded as the hydraulic gradient slope for all the pipes. Table 6-14 shows the results of doing steps 7 through 13 for the entire system.

STRUCTURE NO. (UPPER / LOWER)	LENGTH (ft)	TIME OF CONCENTRATION (MIN)	TIME OF FLOW IN SECTION (MIN)	INTENSITY (iph)	TOTAL C•A	TOTAL FLOW (cfs)	INLET ELEV (ft)	HYD. GRADIENT / CROWN / FLOW LINE: UPPER END ELEV (ft)	HYD. GRADIENT / CROWN / FLOW LINE: LOWER END ELEV (ft)	FALL (ft)	PIPE SIZE (IN) RISE / SPAN	SLOPE (%) HYD GRAD / PHYSICAL / MIN PHYS	ACTUAL VELOCITY (FPS) / PHYSICAL VELOCITY (FPS)
15	300	10	8.5	6.5	0.16	1.04	59.70				18	≈0	.59
								56.7	56.2	0.5		0.167	
14								55.2	54.7		18	.15	2.6
14	300	18.5	1.8	5.1	0.96	4.9	59.80				18	0.18	2.8
								56.2	55.7	0.5		0.167	
13								54.7	54.2		18	0.15	2.6
13	300	20.3	1.3	4.9	1.44	7.05	59.00				18	0.38	4.0
								55.7	51.5	4.2		1.4	
12								54.2	50.0		18	0.15	7.6
12	300	21.6	1.0	4.7	1.84	8.65	54.50				18	0.58	4.9
								51.5	47.5	4.0		1.33	
11								50.0	46.0		18	0.15	7.5
11	300	22.6	-	4.6	2.72	12.5	50.50				18	1.2	7.0
								47.5	46.0	1.5		0.50	
								46.0	44.5		18	0.15	4.5

Table 6-14 Results of First Pass down the System

CALCULATE THE HYDRAULIC GRADE LINE ELEVATION

For pipe section $P_{11\text{-out}}$

14. Determine the Lower End HG elevation. [$P_{11\text{-out}}$]
For the outlet pipe, the Lower End HG is the design tailwater DTW. For this example the design tailwater is the higher of: 1) the crown of the pipe (El. 46.0) or 2) the normal high water stage (47.1) of the lake thus:
Lower End HG = 47.1 ft

15. Determine the Upper End HG elevation. [$P_{11\text{-out}}$]
The Upper End HG is higher of:
1) Lower End HG + full flow friction loss = 47.1 + 1.2% x 300'
= 47.1 + 3.6 = 50.7'
- OR -
2) The elevation of normal depth upstream. The above elevation is higher than the Upper End crown so normal depth cannot control.

Then: Upper End HG = 50.7.

The standard HG clearance is not met (S-11 inlet elev. = 50.5). We will increase this pipe size to 24" before continuing upstream. To match the crowns at the upper end, the flow line of the 24" will be set 0.5' lower than for the 18" pipe.
$P_{11\text{-}out}$ Upper End flow line = 45.5'
Pipe fall = 45.5 - 44.5 = 1 ft (Holding Lower End Flow Line)
Physical Slope = 1 / 300 = 0.33%

Starting at the outlet pipe again:

16. Determine the Lower End HG elevation. [$P_{11\text{-}out}$]
Using the same approach as in step 14, Lower End HG elevation = 47.1

17. Determine the Upper End HG elevation. [$P_{11\text{-}out}$]
The Upper End HG is higher of:
1) Lower End HG + full flow friction loss = 47.1 + 0.26% x 300'
= 47.1 + 0.78 = 47.9'

OR

2) The elevation of normal depth upstream. The above elevation (47.9) is higher than the crown (47.5), so normal depth does not apply.
Then, Upper End HG = 47.9'

Repeat Steps 16 & 17 for the other pipe sections.

<table>
<tr><th rowspan="3">STRUCTURE NO.</th><th rowspan="5">LENGTH (ft)</th><th colspan="3">HYD. GRADIENT</th><th rowspan="2">PIPE SIZE (IN)</th><th rowspan="4">SLOPE (%)</th></tr>
<tr><th colspan="3">CROWN</th></tr>
<tr><th colspan="3">FLOW LINE</th><th>RISE</th></tr>
<tr><th>UPPER</th><th rowspan="2">UPPER END ELEV (ft)</th><th rowspan="2">LOWER END ELEV (ft)</th><th rowspan="2">FALL (ft)</th><th rowspan="2">SPAN</th></tr>
<tr><th>LOWER</th><th>HYD. GRAD. / PHYSICAL / MIN. PHYS.</th></tr>
<tr><td rowspan="2">11</td><td rowspan="3">300</td><td>47.9</td><td>47.1</td><td>0.78</td><td>24</td><td>0.26</td></tr>
<tr><td>47.5</td><td>46.5</td><td rowspan="2">1.0</td><td rowspan="2">24</td><td>0.33</td></tr>
<tr><td>outlet</td><td>45.5</td><td>44.5</td><td>0.10</td></tr>
</table>

Table 6-15

Table 6-16 shows the results of completing these steps for the entire system.

For Pipe Section $P_{12\text{-}11}$:

The Lower End HG is higher of:

1) Downstream pipe Upper End HG: = 47.9'

OR

2) Controlling Pipe Condition at the Lower End. The above elevation (47.9) is higher than the crown (47.5), so normal depth does not apply.

Then, Lower End HG = 47.9'

The Upper End HG is higher of:

1) Lower End HG + full flow friction loss = 47.9 + 0.58% x 300'
= 47.9 + 1.74 = 49.64'

OR

2) The elevation of normal depth upstream. Using the Hydraulic Calculator (Q = 8.65 cfs, 18" pipe @ 1.33% slope), the normal depth = 0.6 x Diameter.
Upper End Normal Depth elev. = 50.0 + 0.6 x 1.5 = 50.9' (d_{NORM} = 0.6 x D)

Then, Upper End HG = 50.9'

For Pipe Section $P_{13\text{-}12}$:

The Lower End HG is higher of:

1) Downstream pipe upper end HG = 50.9'

OR

2) Controlling Pipe Condition at the Lower End. The pipe slope (1.4%) is steeper than the full flow friction slope (0.38%) so, if the downstream HG is low enough, the flow depth at the lower end is normal depth. Thus the controlling pipe condition is normal depth (Figure 5-1C). Using the Hydraulic Calculator (Q = 7.05 cfs, 18" pipe @ 1.4% slope) the normal depth = 0.52 x Diameter.
Lower End Normal Depth elev. = 50.0 + 0.52 x 1.5 = 50.78'

Then, Lower End HG = 50.9'

The Upper End HG is higher of:

1) Lower End HG + full flow friction loss = 50.9 + .38% x 300'
= 50.9 + 1.14 = 52.04'

OR

2) The elevation of normal depth upstream. = 54.2 + 0.52 x 1.5 = 54.98' (d_{NORM} = 0.52 x D)

Then, Upper End HG = 54.98'

For Pipe Section P_{14-13}:

The Lower End HG is higher of:

1) Downstream Pipe Upper End HG = 54.98'

OR

2) Controlling Pipe Condition at the Lower End. The pipe slope (0.167%) is flatter than the full flow friction slope (0.18%) so, use the crown of the pipe (Figure 5-1D) as the controlling pipe condition at the lower end. Lower End Crown elev. = 55.7'

Then, Lower End HG = 55.7'

The Upper End HG is higher of:

1) Lower End HG + full flow friction loss = 55.7 + 0.18% x 300'
= 55.7 + 0.54 = 56.24'

OR

2) The elevation of normal depth upstream. The above elevation (56.24) is higher than the crown (56.2') so normal depth does not apply.

Then, Upper End HG = 56.24'

For Pipe Section P_{15-14}:

The Lower End HG is higher of:

1) Downstream pipe Upper End HG = 56.24'

OR

2) Controlling Pipe Condition at the Lower End. The above elevation (56.24) is higher than the crown (56.2') so normal depth does not apply.

Then, Lower End HG = 56.24'

The Upper End HG is higher of:

1) Lower End HG + full flow friction loss = 56.24 + 0.0' = 56.24'

OR

2) The elevation of normal depth upstream. Using the Hydraulic Calculator (Q = 1.0 cfs, 18" pipe @ 0.17% slope), the normal depth = 0.32 x Diameter. Normal Depth Elevation= 55.2 + 0.32 x 1.5 = 55.68'

Then, Upper End HG = 56.24'

Table 6-16 shows the results of doing steps 16 &17 for the entire system.

STRUCTURE NO. (UPPER / LOWER)	LENGTH (ft)	TOTAL FLOW (cfs)	INLET ELEV (ft)	HYD. GRADIENT / CROWN / FLOW LINE: UPPER END ELEV (ft)	LOWER END ELEV (ft)	FALL (ft)	PIPE SIZE (IN): RISE / SPAN	SLOPE (%): HYD. GRAD. / PHYSICAL / MIN. PHYS.
15	300	1.04	59.70	56.24	56.24		18	≈0
				56.7	56.2	0.5		0.167
14				55.2	54.7		18	0.15
14	300	4.9	59.80	56.24	55.7	0.54	18	0.18
				56.2	55.7	0.5		0.167
13				54.7	54.2		18	0.15
13	300	7.05	59.00	54.98	50.9		18	0.38
				55.7	51.5	4.2		1.4
12				54.2	50.0		18	0.15
12	300	8.65	54.50	50.9	47.9		18	0.58
				51.5	47.5	4.0		1.33
11				50.0	46.0		18	0.15
11	300	12.5	50.50	47.9	47.1	0.78	24	0.26
				47.5	46.5	1.0		0.33
outlet				45.5	44.5		24	0.1

Table 6-16 Results of First Pass up the System

The HG slopes shown for pipe sections $P_{15\text{-}14}$, $P_{13\text{-}12}$, $P_{12\text{-}11}$ are the full flow friction slopes. The values are not the true HG slopes because these pipes are flowing part full. The values will be revised in subsequent iterations through the system. The full flow friction slopes have been shown in Table 6-16 to help follow the discussion of Steps 16 & 17 for the entire system.

18. COMPARE THE HYDRAULIC GRADIENT ELEVATION TO THE STANDARD
Throughout the system, the hydraulic gradient elevation is more than 1.13' below the inlet elevation (edge of pavement) so it meets the current standard. We will recalculate the flow rates and check again.

19. RECALCULATE THE FLOW RATES
Several pipes are flowing partly full, so we need to recalculate the velocities and times of flow in section. This will change the times of concentration and the flow rates. Pipe sections $P_{15\text{-}14}$, $P_{13\text{-}12}$, & $P_{12\text{-}11}$ are flowing part full and the others are flowing full based on the calculations up to this point. We will assume these modes of flow as we work downstream recalculating flow. The velocity in the three pipes flowing partly full will be based on normal depth velocity (see Figure 5-3). Table 6-17 shows the results of recalculating the flow rates.

STRUCTURE NO.	LENGTH (ft)	TIME OF CONCEN-TRATION (min)	TIME OF FLOW IN SECTION (min)	INTENSITY (iph)	TOTAL C•A	TOTAL FLOW (cfs)	PIPE SIZE (IN)	SLOPE (%)	ACTUAL VELOCITY (FPS)	NOTES AND REMARKS
										ZONE: 7
										FREQUENCY (Yrs): 3
										MANNINGS “n”: 0.012
							RISE	HYD. GRAD.	PHYSICAL VELOCITY (FPS)	TAILWATER EL. (ft): 47.1
UPPER								PHYSICAL		All overland $t_c \leq 10$ min.
LOWER							SPAN	MIN PHYS		
15	300	10.0	2.4	6.5	0.16	1.04	18	≈ 0	2.1	Act Vel based on normal Depth. d/D = 0.32
								0.167		
14							18	.15	2.6	
14	300	12.4	1.5	6.0	0.96	5.76	18	0.25	3.25	
								0.167		
13							18	0.15	2.6	
13	300	13.9	0.6	5.7	1.44	8.2	18	0.5	8.0	Act Vel based on normal Depth. d/D = 0.55
								1.4		
12							18	0.15	7.6	
12	300	14.5	0.6	5.6	1.84	10.3	18	0.8	8.3	Act Vel based on normal Depth. d/D = 0.67
								1.33		
11							18	0.15	7.5	
11	300	15.1	-	5.6	2.72	15.2	24	0.38	4.8	
								0.33		
Outlet							24	0.10	4.5	

Table 6-17 Results of Second Pass down the System

20. RECALCULATE THE HYDRAULIC GRADIENT ELEVATION
Work up the system as was done previously in Steps 16 & 17. Table 6-18 shows the results.

STRUCTURE NO.	LENGTH (ft)	TOTAL FLOW (cfs)	INLET ELEV. (ft)	HYD. GRADIENT / CROWN / FLOW LINE: UPPER END ELEV. (ft)	LOWER END ELEV. (ft)	FALL (ft)	PIPE SIZE (IN)	SLOPE (%)	NOTES AND REMARKS
									ZONE: 7
									FREQUENCY (Yrs): 3
									MANNING’S “n”: 0.012
							RISE	HYD GRAD	TAILWATER EL. (ft): 47.1
UPPER								PHYSICAL	
LOWER							SPAN	MON PHYS	All overland $t_c \leq 10$ min.
15	300	1.04	59.70	56.45	56.45		18	≈ 0	Act.Vel based on Norm Depth. d/D = 0.32
				56.7	56.2	0.5		0.167	
14				55.2	54.7		18	0.15	
14	300	5.76	59.80	56.45	55.7	0.75	18	0.25	
				56.2	55.7	0.5		0.167	
13				54.7	54.2		18	0.15	
13	300	8.02	59.00	55.03	51.0		18	0.5	Act Vel & Upper End HG Based on Norm Depth D/D = 0.55
				55.7	51.5	4.2		1.4	
12				54.2	50.0		18	0.15	
12	300	10.3	54.50	51.0	48.24		18	0.8	Act Vel & Upper End HG Based on Norm Depth D/D = 0.67
				51.5	47.5	4.0		1.33	
11				50.0	46.0		18	0.15	
11	300	15.2	50.50	48.24	47.1	1.14	24	0.38	
				47.5	46.5	1.0		0.33	
OUT				45.5	44.5		24	0.10	

Table 6-18 Results of Second Pass up the System

The HG slopes shown for pipe sections $P_{15\text{-}14}$, $P_{13\text{-}12}$, $P_{12\text{-}11}$ are the full flow friction slopes. The values are not the true HG slopes because these pipes are flowing part full. The full flow friction slopes have been shown in Table 6-18 to help you compare HG elevations as you work through the system. The values are changed in Table 6-19, which reflects the completed design.

21. COMPARE THE HYDRAULIC GRADIENT TO THE STANDARD

Throughout the system, the hydraulic gradient elevation is more than 1.13' below the inlet elevation (edge of pavement) so it meets the current standard. Pipe section $P_{11\text{-}out}$ cannot be reduced in diameter without violating the standard HG clearance at S-11 (see step 15). The other pipes are the minimum standard diameter so their diameter cannot be reduced.

Pipe section $P_{15\text{-}14}$ is flowing full for about ½ of its length. Consequently the flow velocity is less than the 2.1 fps we estimated in Table 6-17. We could make another iteration through the system recalculating flows based on the reduced velocity, but there is nothing to be gained from doing that here. None of the pipe diameters can be reduced.

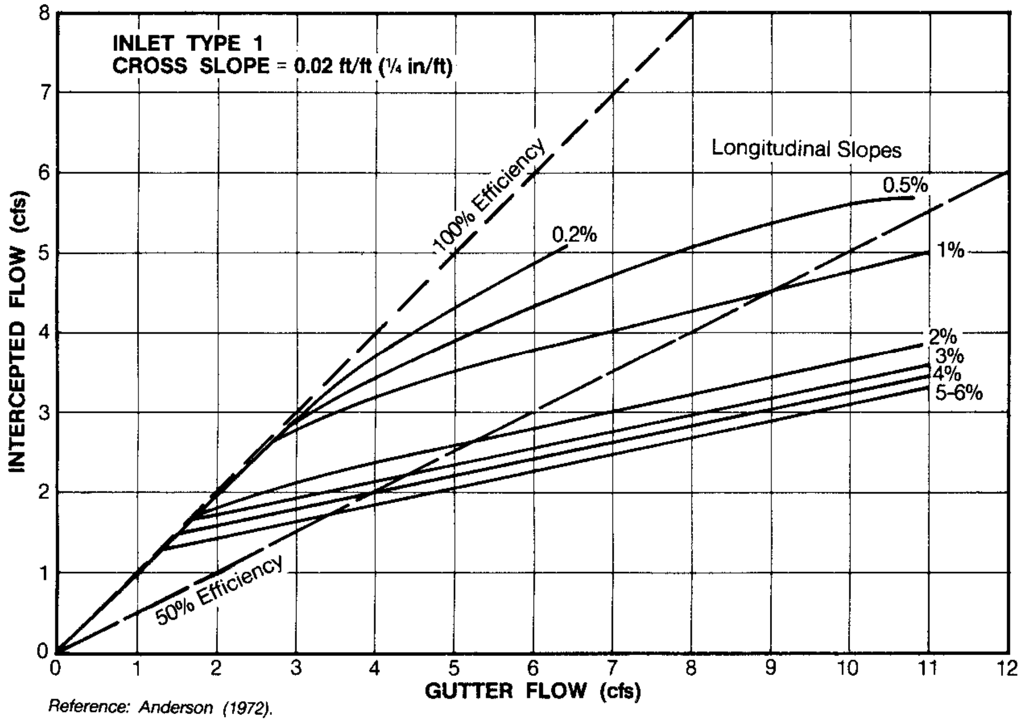
A completed tabulation form is shown in Table 6-19.

Example 6.2

LOCATION OF UPPER END ALIGNMENT NAME STATION	DISTANCE (ft)	SIDE	STRUCTURE NO. UPPER LOWER	TYPE OF STRUCTURE	LENGTH (ft)	DRAINAGE AREA (ac) C = C = 0.80 C = INCRE-MENT	TOTAL	SUBTOTAL C•A	TIME OF CONCEN-TRATION (min)	TIME OF FLOW IN SECTION (min)	INTENSITY (iph)	TOTAL C•A	TOTAL FLOW (cfs)	INLET ELEV (ft)	HYD. GRADIENT CROWN FLOW LINE UPPER END ELEV (ft)	LOWER END ELEV (ft)	FALL (ft)	PIPE SIZE (IN) RISE SPAN	SLOPE (%) HYD GRAD PHYSICAL MIN PHYS	ACTUAL VELOCITY (FPS) PHYSCIAL VELOCITY (FPS)	NOTES AND REMARKS ZONE: 7 FREQUENCY (Yrs): 3 MANNING'S "n": 0.012 TAILWATER EL. (ft): 47.1 All overland $t_c \le 10$ min
Patrick Place			15	P1	300	0.2	0.2	0.16	10	2.4	6.5	0.16	1.04	59.70	56.45	56.45	≈ 0	18	≈ 0	2.1	Act Vel based on Norm Depth. d/D = 0.32
															56.7	56.2	0.5		0.167		
			14												55.2	54.7		18	0.15	2.6	
Patrick Place			14	P1	300	1.0	1.2	0.96	12.4	1.5	6.0	0.96	5.76	59.80	56.45	55.7	0.75	18	0.25	3.25	
															56.2	55.7	0.5		0.167		
			13												54.7	54.2		18	0.15	2.6	
Patrick Place			13	P1	300	0.6	1.8	1.44	13.9	0.6	5.7	1.44	8.2	59.00	55.03	51.0	4.03	18	1.3	8.0	Act Vel & Upper End HG based on Norm Depth. d/D = 0.55
															55.7	51.5	4.2		1.4		
			12												54.2	50.0		18	0.15	7.6	
Patrick Place			12	P1	300	0.5	2.3	1.84	14.5	0.6	5.6	1.84	10.3	54.50	51.0	48.24	2.79	18	0.9	8.3	Act Vel & Upper End HG based on Norm Depth. d/D = 0.67
															51.5	47.5	4.0		1.33		
			11												50.0	46.0		18	0.15	7.5	
Patrick Place			11	P1	300	1.1	3.4	2.72	15.1	1.0	5.6	2.72	15.2	50.50	48.24	47.1	1.14	24	0.38	4.8	
															47.5	46.5	1.0		0.33		
			OUT												45.5	44.5		24	0.10	4.5	

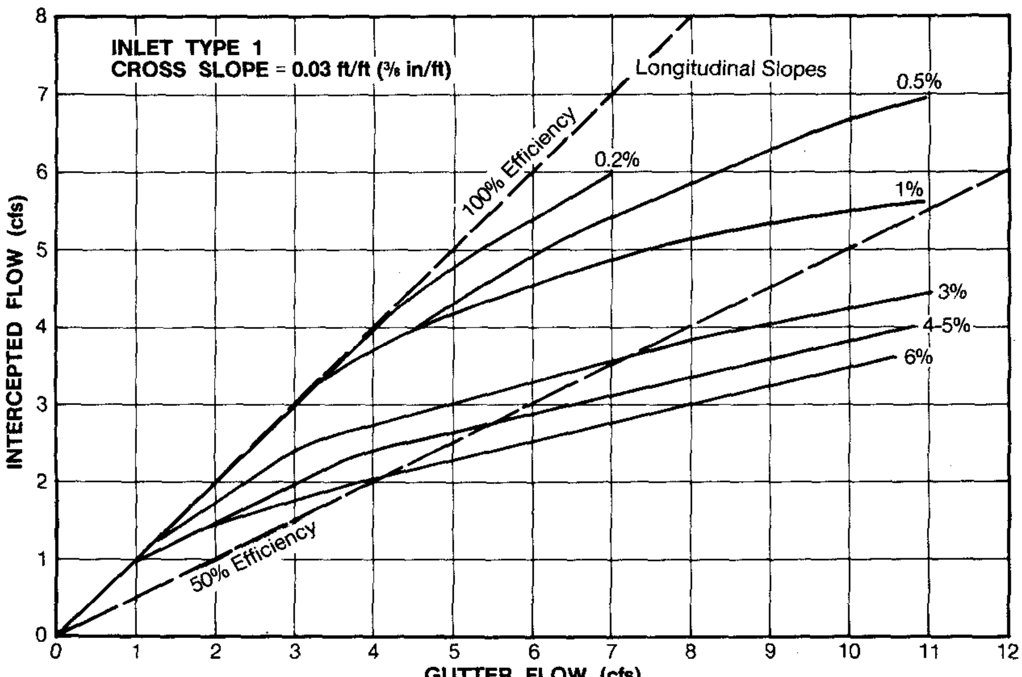
Table 6-19

Appendix A - Inlet Efficiencies



Reference: Anderson (1972).

Figure A-1



Reference: Anderson (1972).

Figure A-2

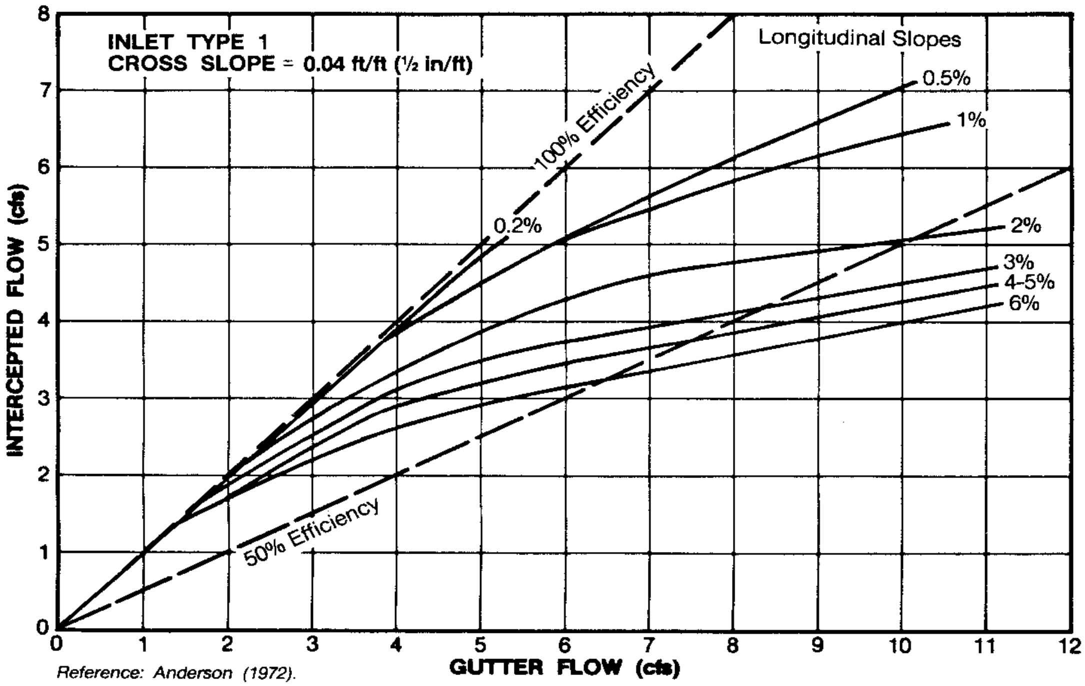


Figure A-3

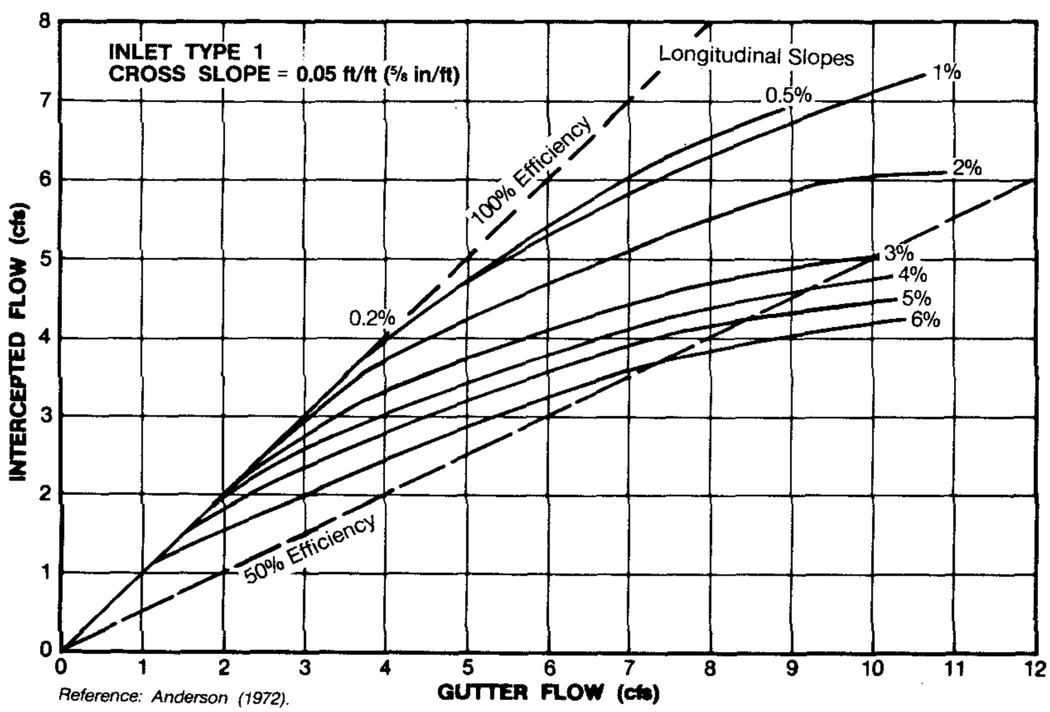


Figure A-4

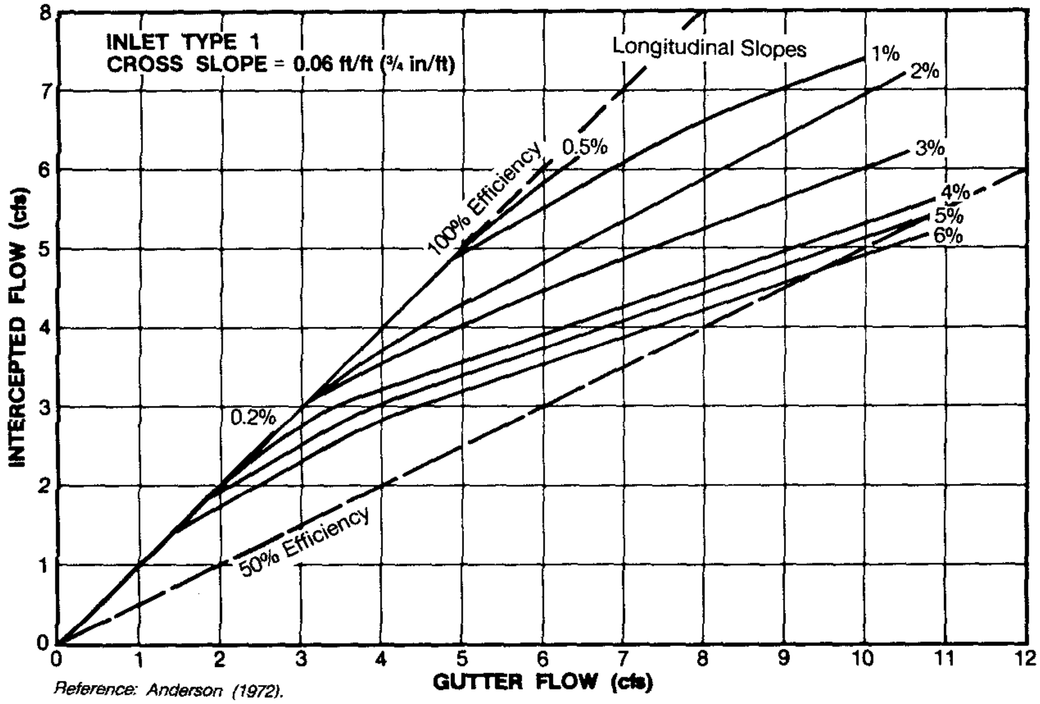


Figure A-5

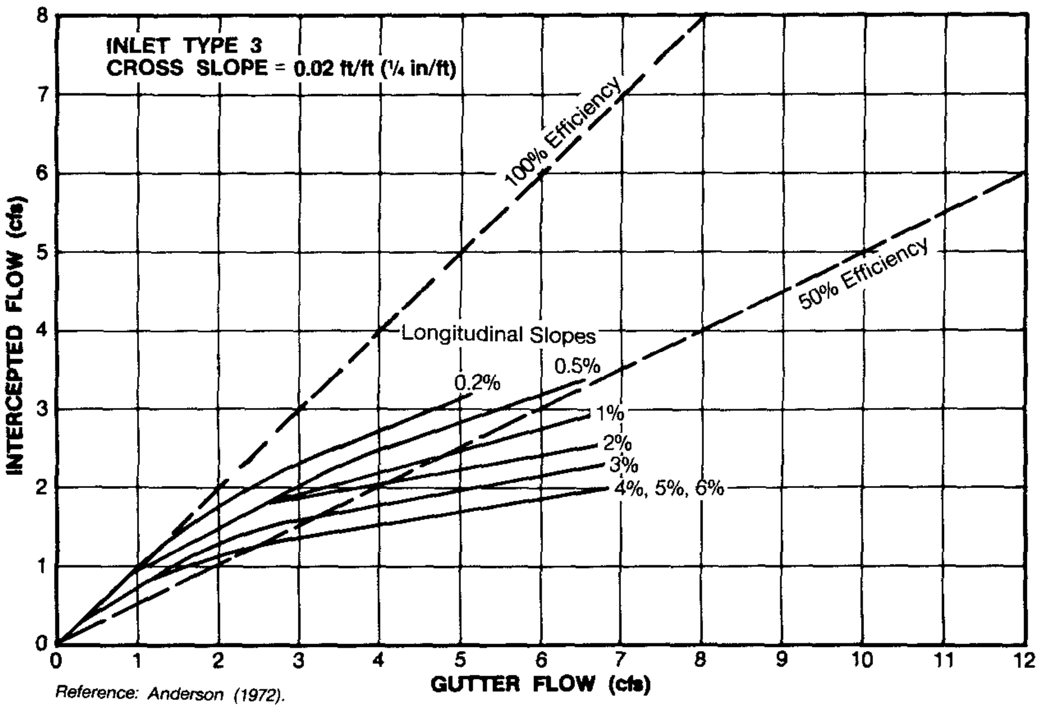


Figure A-6

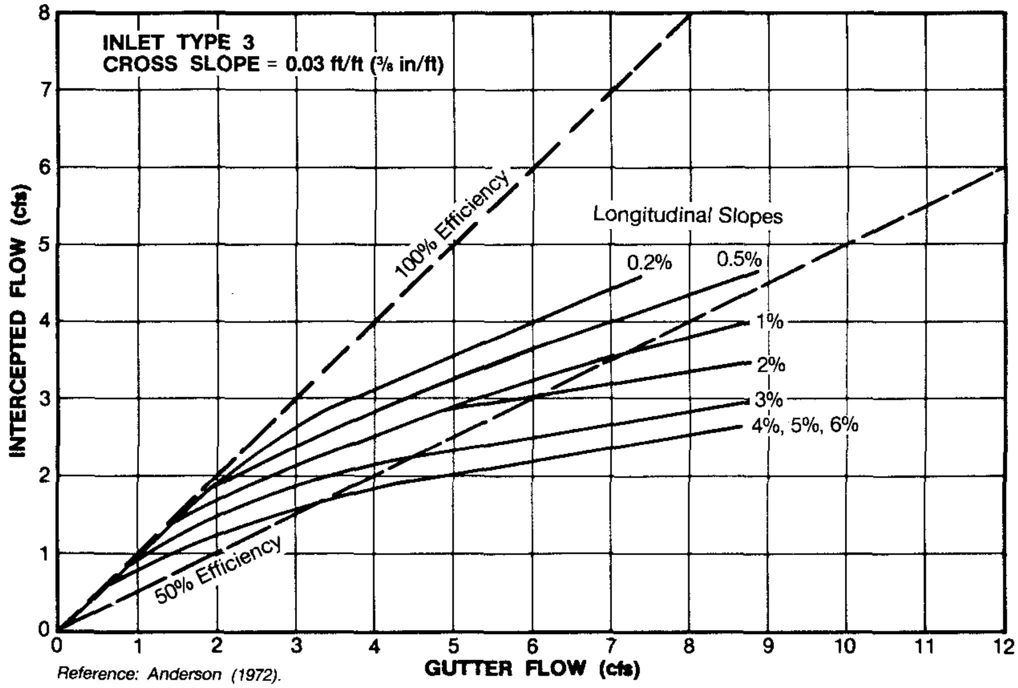


Figure A-7

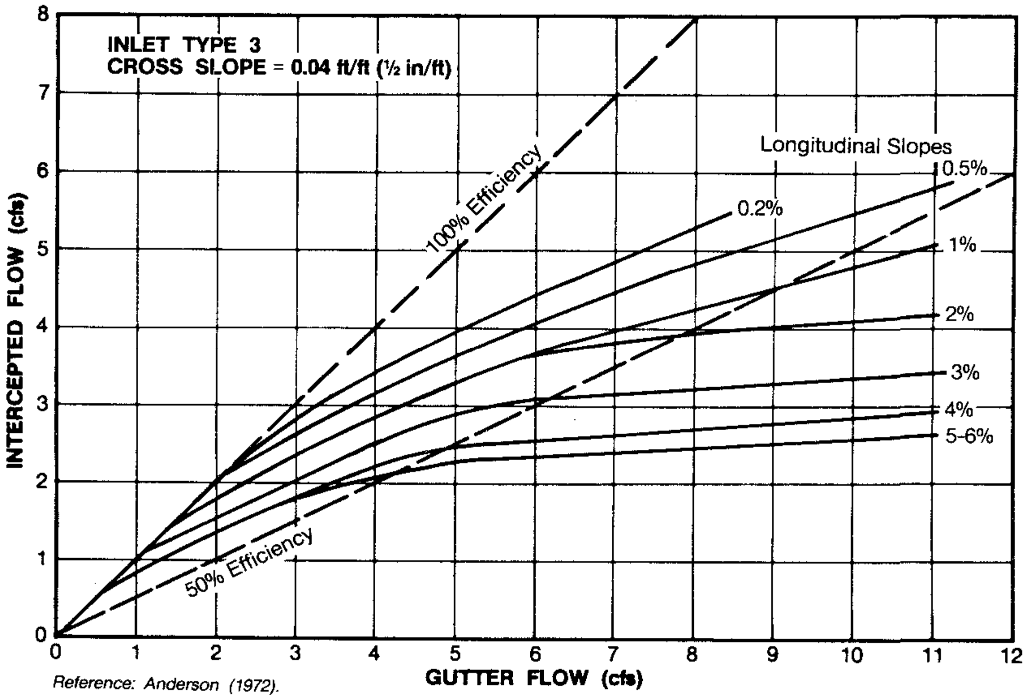


Figure A-8

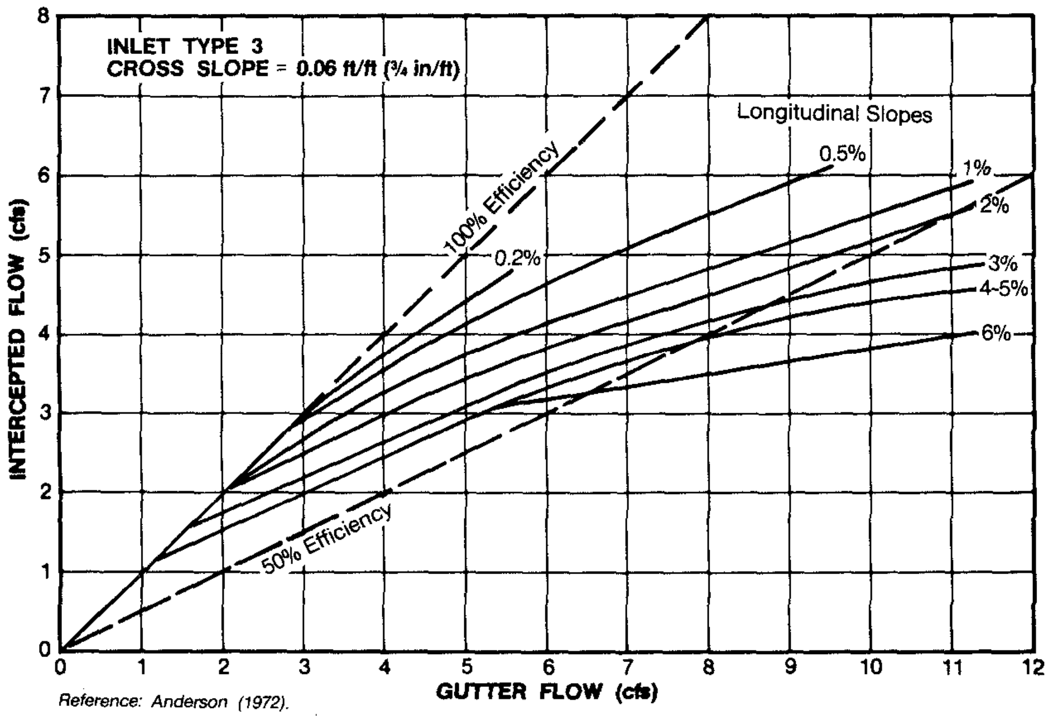


Figure A-9

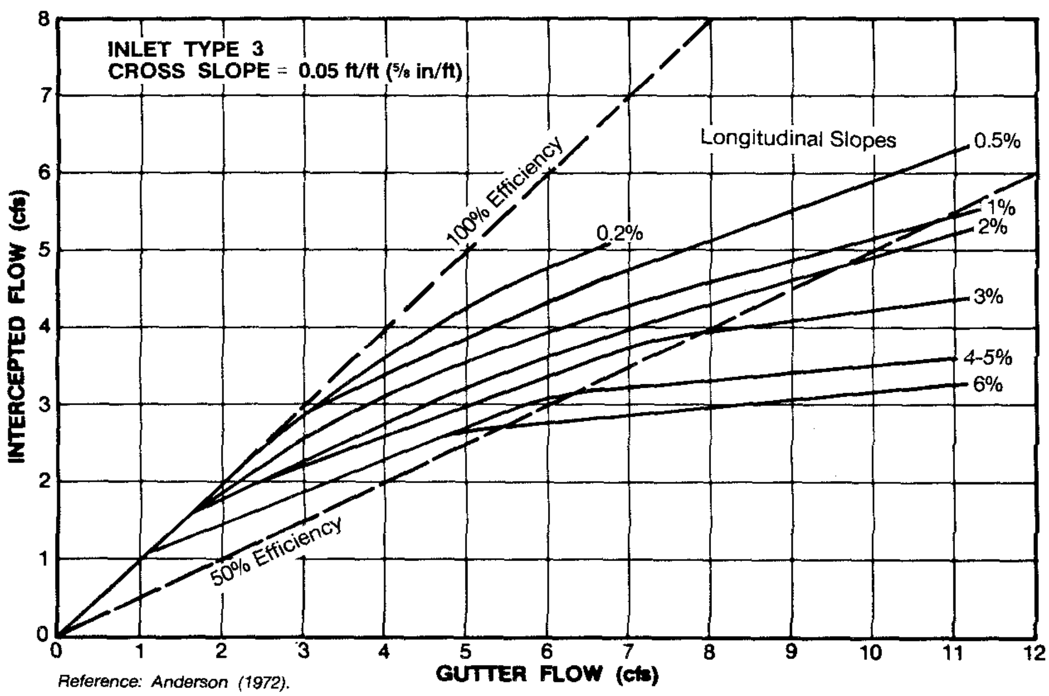
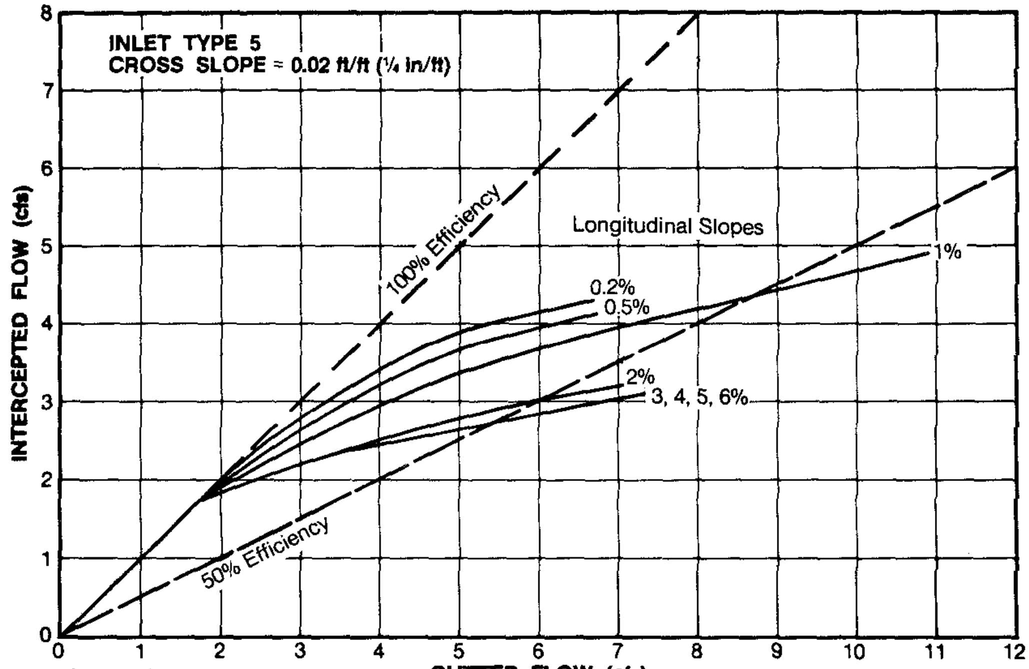
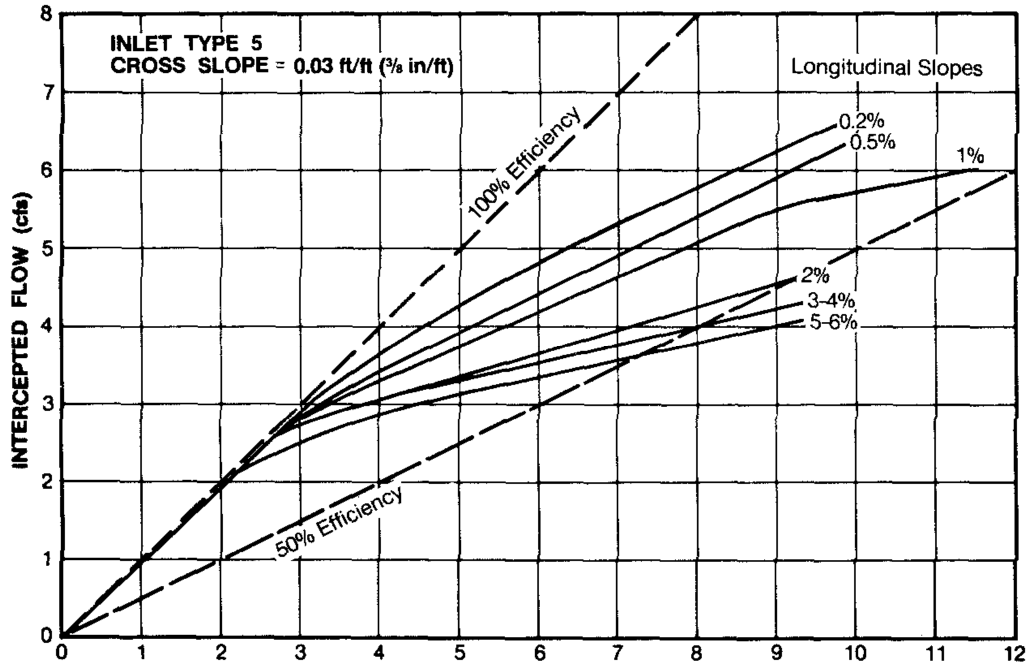


Figure A-10



Reference: Anderson (1972).

Figure A-11



Reference: Anderson (1972).

Figure A-12

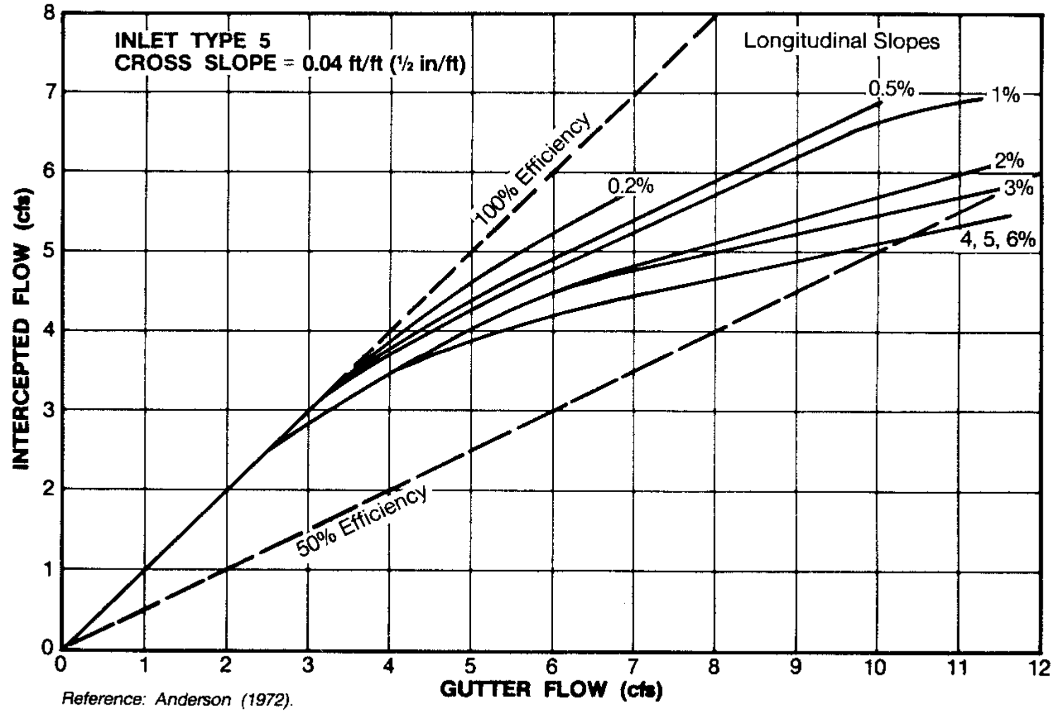


Figure A-13

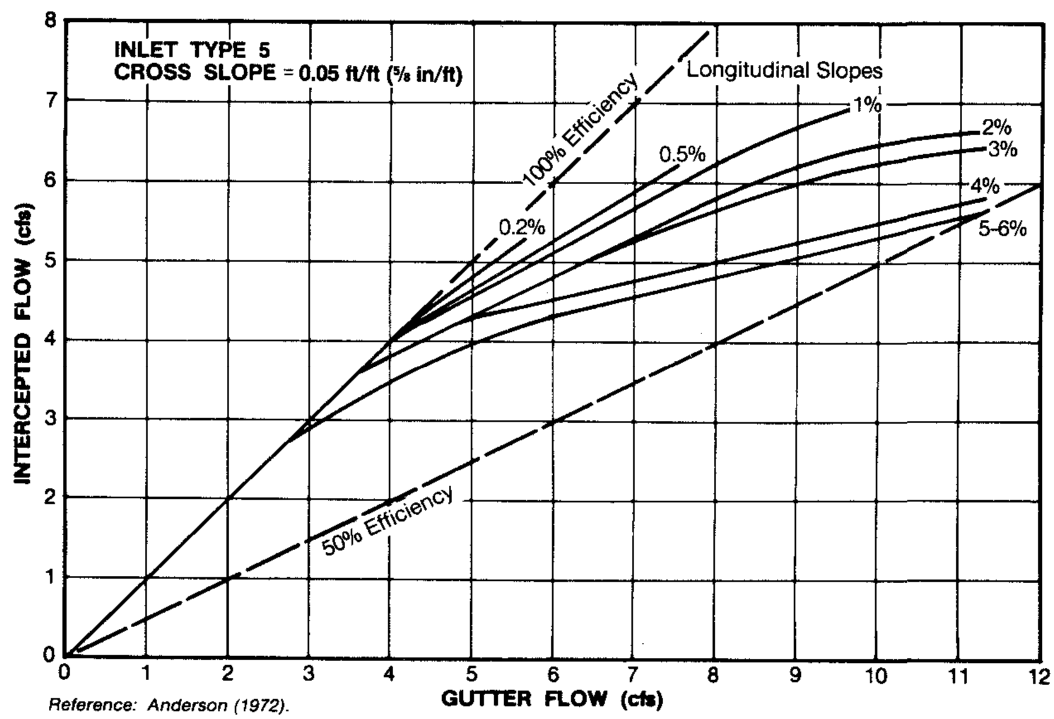
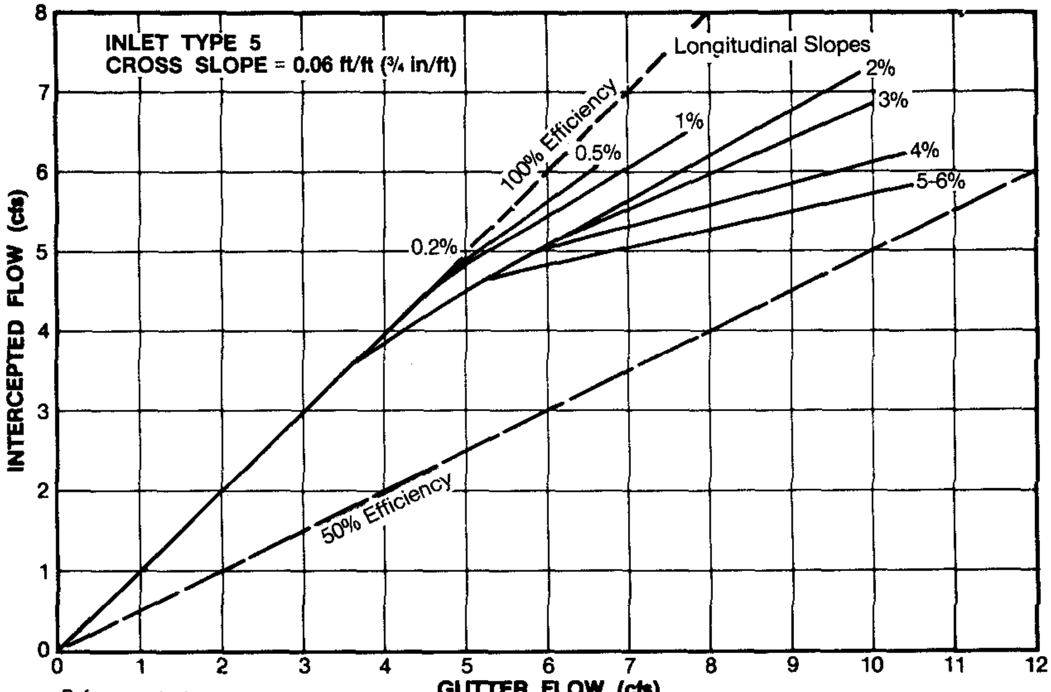
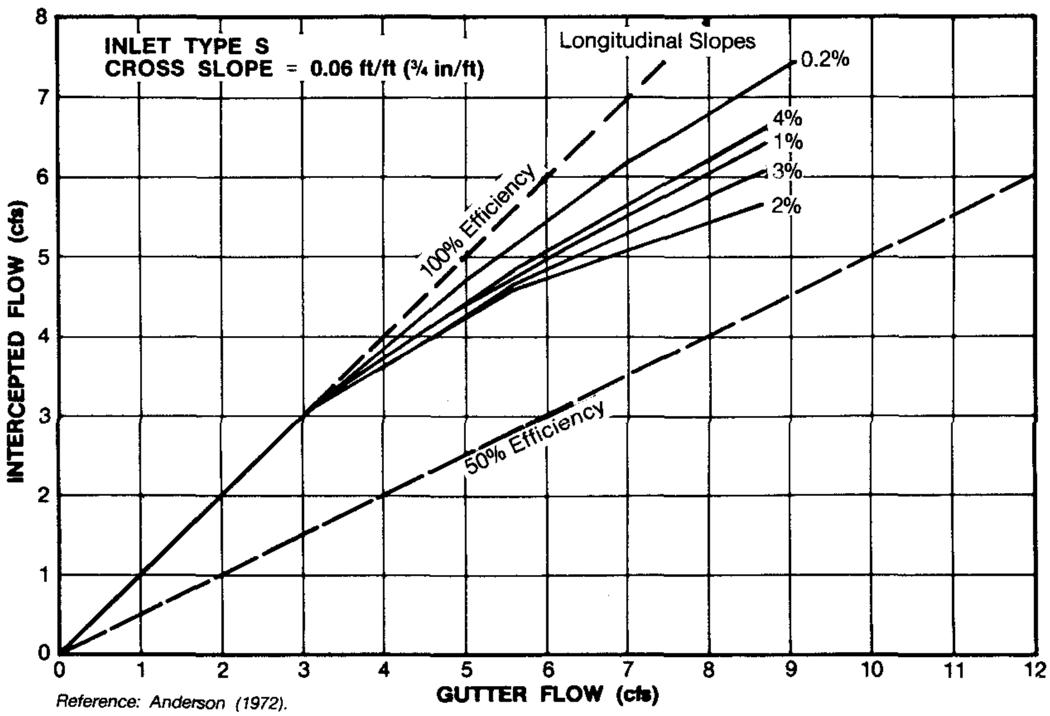


Figure A-14



Reference: Anderson (1972).

Figure A-15



Reference: Anderson (1972).

Figure A-16

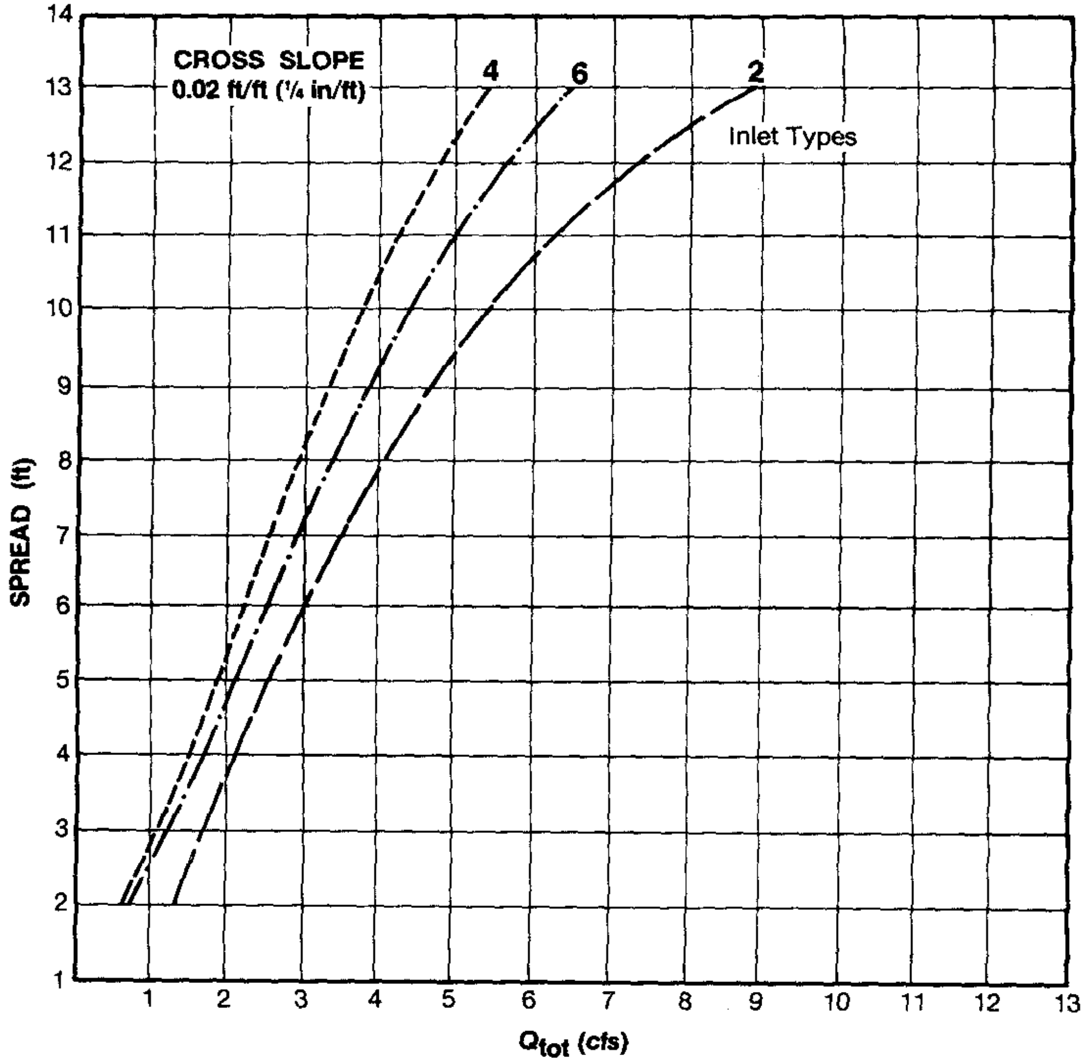


Figure A-17, Sump Conditions for Types 2, 4 & 6

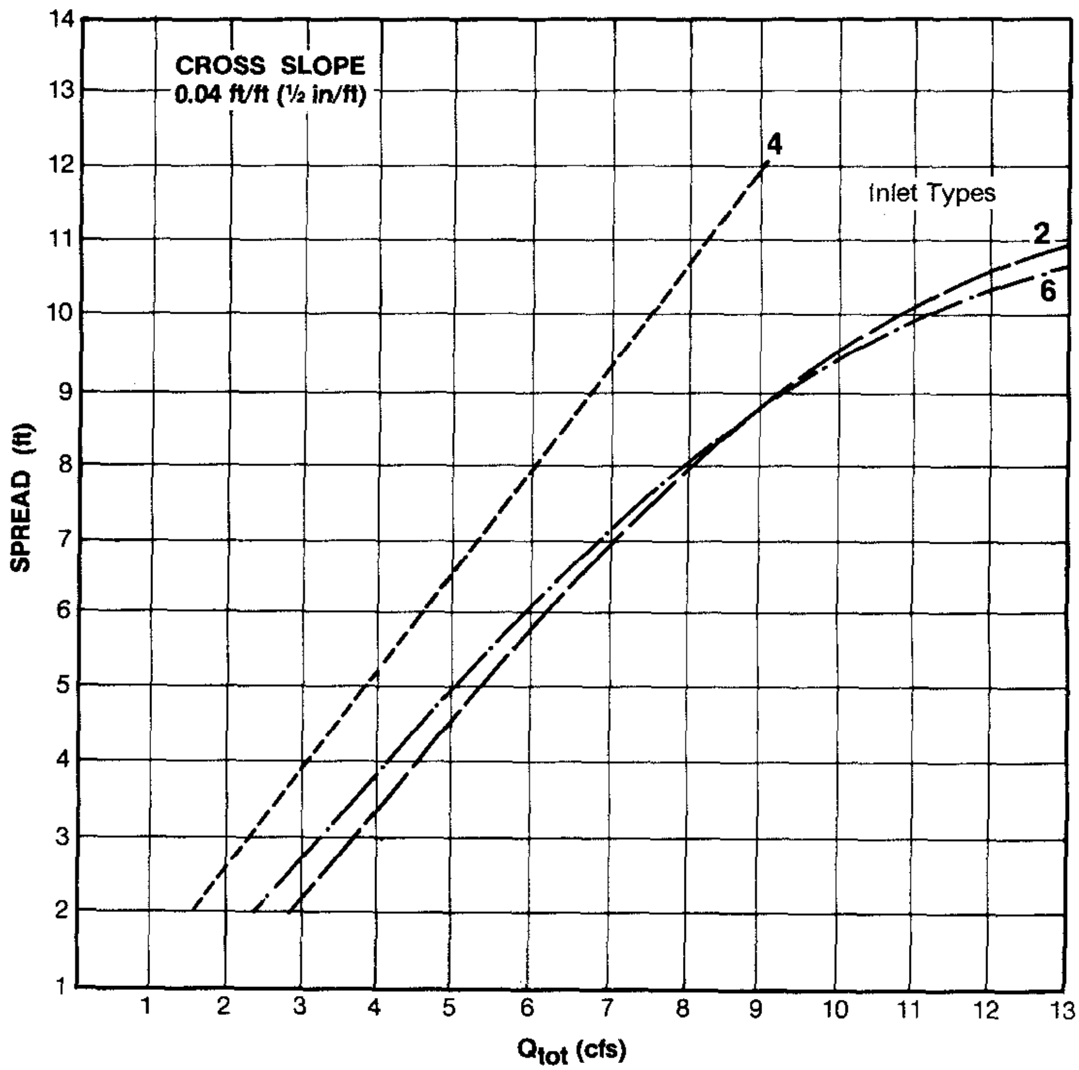


Figure A-18, Sump Conditions for Types 2, 4 & 6

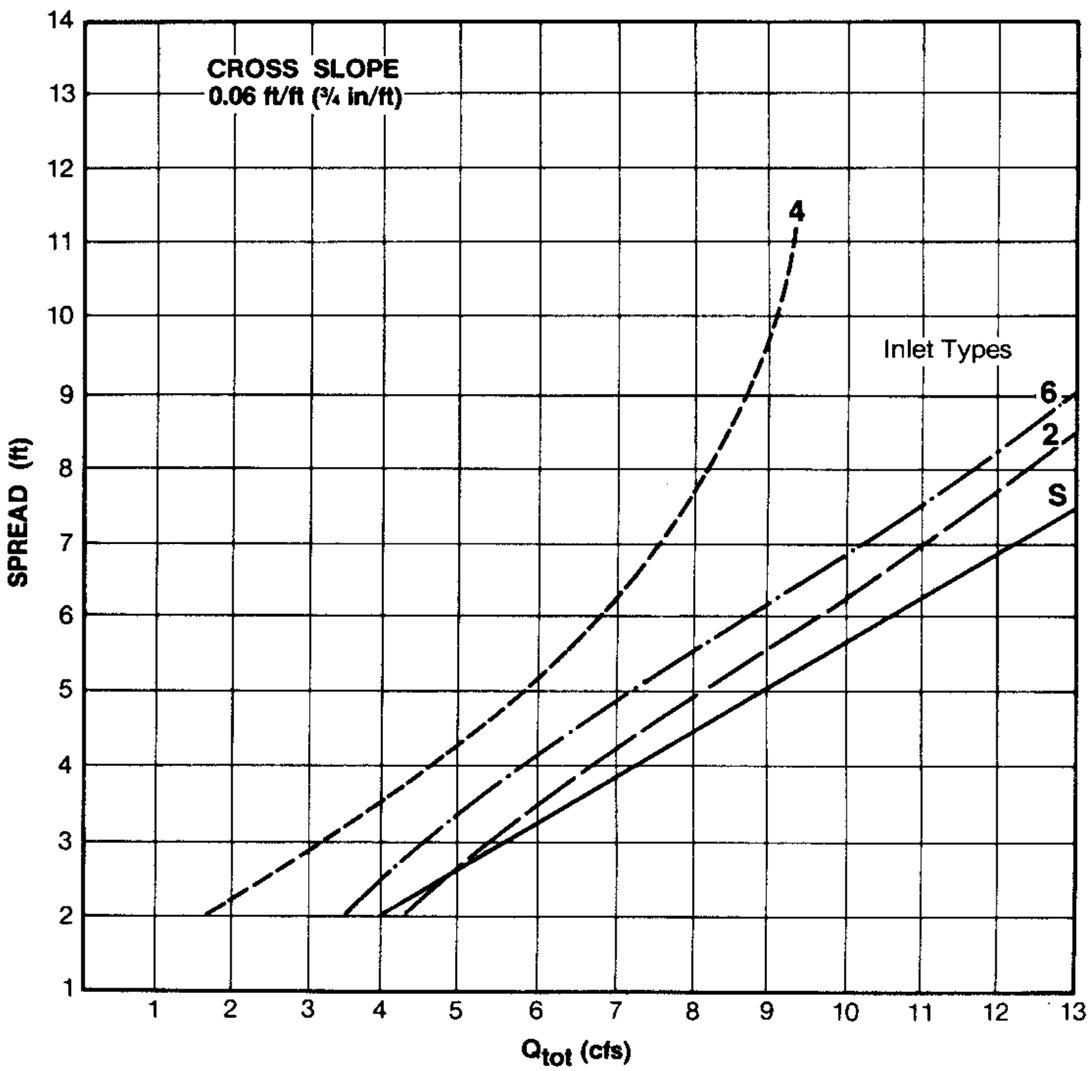


Figure A-19, Sump Conditions for Types 2, 4, 6 & S

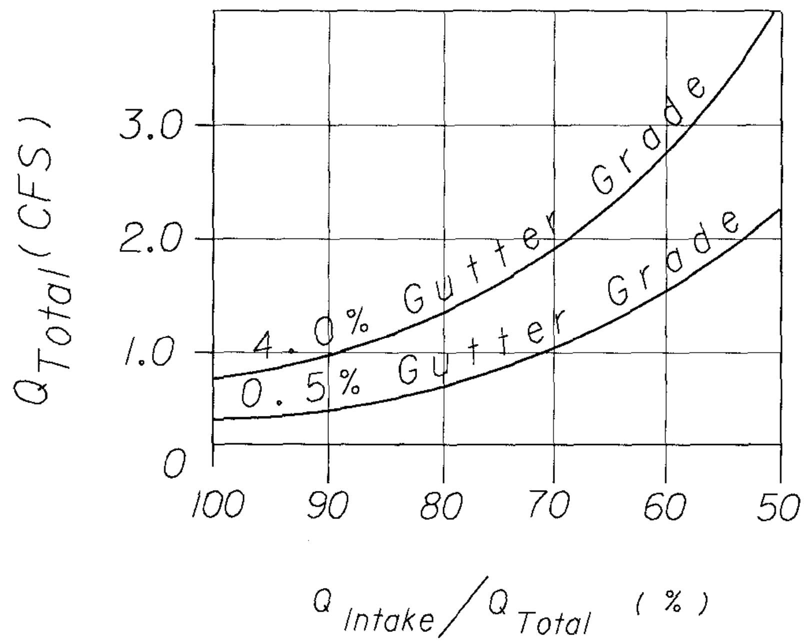


Figure A-20, Type 9 Inlet

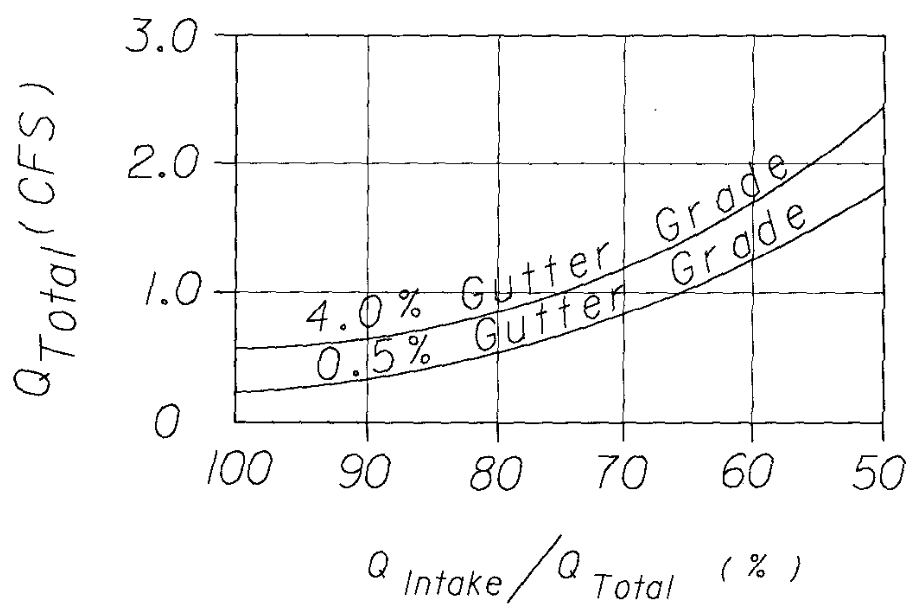
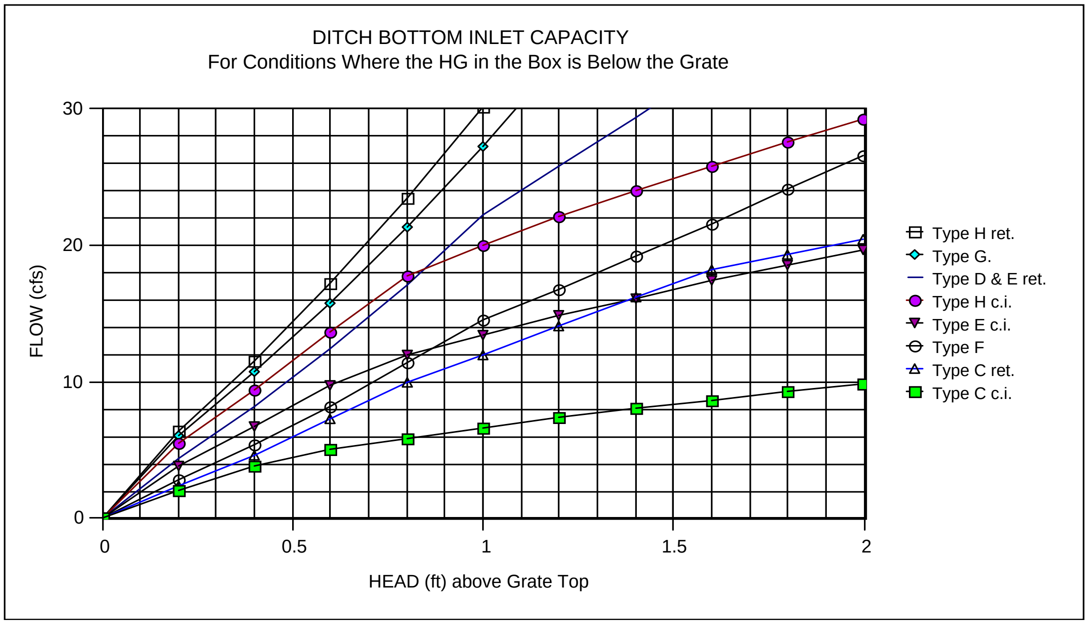


Figure A-21, Type 10 Inlet



1. The above graph should be used where the hydraulic gradient in the inlet box is below the top of the grate. For other conditions, see the discussion below.
2. The above is based on 50% debris blockage and inlets without slots.
3. The symbols on the curves do not represent-measured data points. They are calculated points from the equation and coefficients in the research report by the University of South Florida titled “Investigation of Discharge through Grated Inlets”, February 1993, WPI No. 0510611. Contact the FDOT Research Center at 850-414-4615 to obtain a copy. The grate flow areas used in the equations are from U.S. Foundry & Mfg. Corp.

Where the hydraulic gradient is above the top of the grate, the system capacity may control the flow through the grate. The total system loss is a sum of friction losses and various minor losses, including the loss across the grate. In this situation, the loss across the grate is typically small but can be calculated from:

$$\text{Head Loss [ft]} = K\frac{V_g^{\,2}}{2g}$$

Where K = 0.46 for reticuline grates; 3.2 for cast iron grates
V_g = velocity [fps] across the grate based on the grate full face area (grate width x grate length)
g = acceleration of gravity

Example:
A DBI is needed to capture 5 cfs in a depressed area behind the sidewalk. The hydraulic gradient due to friction loss in the system is estimated to be 0.8 ft above the grate.

Try a Type C DBI:
Full Face Area = 2.33' x 3.0' = 7.0 ft^2,
then V_g =Q / A = 5 / 7 = 0.7 fps
Assume a Cast iron grate is used. Then K = 3.2
Then: Head loss $= K \times V_g^2/2g = 3.2 \times (0.7)^2 / 64.4 = 0.02$ ft

This is an insignificant amount of head loss and is typical of most design situations. Where a DBI accepts high flow rates (usually under high head conditions) as perhaps in a stormwater pond, the additional loss could be substantial and may dictate a larger inlet (large grate area.)